AN INTEGRATED TECHNIQUE FOR THE ANALYSIS OF FRAME AND CABLE STRUCTURES

By

PETROS MARCOU CHRISTOU

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To my family

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Petros Marcou Christou

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Chairman: Marc I. Hoit
Cochairman: Ronald A. Cook
Major Department: Civil Engineering

The analysis of Frame and Cable structures has always been a challenging problem for the structural engineer. This dissertation provides a solution strategy for the analysis of Frame and Cable structures which are defined as cable networks supported by a space frame. The solution strategy is an iterative technique and consists of the Force Density Method for the analysis of cable networks and the Direct Stiffness Method. The two methods are performed successively to obtain an approximate solution for the structure under consideration. When the approximate solution for the structure is obtained, a nonlinear variation of the Direct Stiffness Method which includes large displacements is used to obtain the final solution.

This dissertation also provides the implementation of a true cable element in the Force Density Method. The implementation of such element allows for the better

modeling of the cable elements which do not have the ability to support any compressive forces. Rather they become slack and stay under tension of their own self weight.

The solution strategy is applied to the analysis of the cable supported signal lights and signs that are used in the state of Florida. The computer program ATLAS was developed to perform the analysis of these structures. ATLAS considers the different kinds of loads on the structures (gravity, wind) and provides the stresses in the individual elements.

The results of the analytical solution showed a very good correlation with the results of static load tests which were performed at the University of Florida Structures Laboratory.

CHAPTER 1
INTRODUCTION - LITERATURE REVIEW

1.1 Introduction

Cables distinguish themselves in that they can be packaged in a compact volume and yet retain the same structural properties as rod elements when extended. They function as tension truss members when in a stretched state and they cannot support any compressive force. The applications which incorporate cable elements are therefore very interesting and knowledge of their analysis and behavior is a challenge to the structural engineer. A Frame-Cable structure (FC) is defined as a pre-stressed cable network which is supported by a space frame. This dissertation is concerned with the analysis of FC structures using a new technique developed by the author which consists of two other methods; the Force Density Method for cable network analysis and the more familiar Direct Stiffness Method. The resulting work includes an extension to the Force Density Method to handle cables which may become slack under the application of a certain type loading.

1.2 Problem Statement

The State of Florida has always been a target of high winds and hurricanes. Hurricane Andrew, which hit South Florida on August 1992, caused excessive damage in airports, buildings and essentially all types of structures. Following the disaster, an inspection and damage evaluation was performed by the Department of Civil Engineering at the University of Florida, regarding the cable-supported signals and signs. The most

striking observation during the inspection/damage evaluation was the poor performance of the cable-supported signs and signals. According to the report [1] submitted to the Florida Department of Transportation (FDOT), essentially all such structures were damaged over an area of approximately 400 square miles; damage on the cable-supported signals and signs were reported even in areas where no other structures or vegetation were damaged.

1.3 Need for the Study

The operability of traffic signal lights and signs used in highway applications is critical in emergency evacuations and post-disaster recovery periods. These systems must support the applied loads especially those applied due to wind, and remain intact after physical disasters, i.e. hurricanes. The absence of any reliable analysis technique for the cable supported signal lights and/or signs, leads to the under-estimation of the forces that are developed in the pole, cable and connector elements. As a result, these structures suffered excessive damages when they were subjected to loads induced by hurricane winds. The need for the present study is justified by the magnitude of the damage that these systems suffered during Hurricane Andrew.

1.4 Scope

This dissertation presents an integrated technique for the analysis of Frame-Cable structures. The technique is implemented in a computer program that is used for the analysis of different signal lights' and/or signs' supports of different configurations which consist of pre-stressed concrete or steel poles, and cables. The signal lights and/or signs are attached to these supports. The analysis technique consists of a combination of the force density method for cable networks, and the direct stiffness method. The gravity

loads and the wind loads are the governing loading patterns on these structures. The nature of the applied load (wind) can cause different cable elements to go slack. The technique presented herein models the true cable behavior including slack cables.

1.5 Objectives

The objectives of this study are as follows:

1. Implement the Force Density Method with necessary external constraints (Node distance conditions, Unstrained length) for the analysis of cable networks.
2. Implement the Direct Stiffness Method for framed structures. The method is extended to nonlinear to include large displacements and geometric nonlinearities.
3. Develop a technique that consists of the two methods (Force Density, Direct Stiffness) which can be used for the analysis of Frame-Cable structures.
4. Extend the Force Density Method to model true cable elements with the ability to go slack.
5. Develop a computer program which implements this technique and it is used for the analysis of several configurations for the signal light and/or sign supports.

1.6 Summary

The analysis of cable networks has always been a challenging problem for the structural engineer. Chapter 2 provides the theoretical background of the Force Density Method which can be used for the analysis of cable networks. Cables can go slack; any cable network analysis would not be powerful unless it is able to handle slack cables. The extension to the Force Density Method to handle true cable behavior (including slack cables), is presented in Chapter 3. The Direct Stiffness Method has been the most commonly used method for the analysis of regular framed structures. The ability of such

structures to undergo large displacements requires a nonlinear variation of the Direct Stiffness Method to account for geometric nonlinearities. Chapter 4 provides an overview of the Nonlinear Direct Stiffness Method which can be used for the analysis of nonlinear systems. It is shown throughout this study that the two aforementioned methods can be combined for the analysis of Frame-Cable structures. The resulting technique as well as the computer program which was developed to implement this technique are presented in Chapter 5. Chapter 6 provides several examples of Frame-Cable structures that can be analyzed using the technique that is presented in Chapter 5, and, finally, Chapter 7 presents some conclusions from this study as well as some useful recommendations for future work in a related subject.

1.7 Literature Review

A Frame-Cable (FC) structure is a broad category of structures which range from very small applications such as a tennis racket to very large and complex applications such as cable suspended roofs and space structures. The analysis of such structures is not always an easy task since traditional methods of structural analysis such as the linear direct stiffness method cannot be applied directly. The level of difficulty for the exact analysis is even higher when the cables form a network. Difficulties in the analysis arise from both the material and geometric nonlinearities of cable networks. The material nonlinearity is due to cable slackening, while the geometric nonlinearity stems from the large displacements and geometric stiffness due to the applied tensions [2]. Slackening may, however, be treated as a beam buckling, which is a geometric nonlinearity. In addition to these, the extreme sensitivity in the stress pattern in the cable network structures to slight changes of the geometry of the cables is a complicating factor. The difficulty is enhanced even more when dealing with FC structures where the frame

deformations have to be accounted for as well. A FC structure is different from a conventional structure, such as a space truss, in that its initial shape is stabilized by the prestress in the cables. Analysis of such structures includes shape finding and loading analysis. Shape finding of a FC structure is a process of obtaining the initial equilibrium configuration or initial shape with a given set of requirements. These requirements may consist of such aspects as the connectivity pattern of the structure, its approximate overall dimensions, the support reactions and the initial stresses in the cables. Loading analysis of a FC structure is the process of finding the member stresses and structural deflections under external loads with respect to the initial shape. In most FC structure applications, the results of the two stages of analysis can be superimposed. The use of the superposition method implies that the frame members are regarded as linear elements in the analysis. In the engineering design of a FC structure this assumption is justified only when small deflections of the supporting frame are allowed to occur. Therefore it is important to establish a design criterion as to whether the interaction between the cables and frame members should be considered [3]. There are cases that the assumption of small frame displacements is not valid. In those cases the displacements of the space frame should be taken into account, and misjudgment of both the initial shape of a FC structure and its loading capacity will result, if the integrated analysis is not adopted.

Cable networks and essentially FC structures can, theoretically, be analyzed using any method developed for other structures. However, the material and geometric nonlinearities create numerical difficulties in solving the equations for the analysis of such structures. Several methods were developed to overcome the aforementioned difficulties and solve the required equations. These methods can be divided in two major categories based on the assumptions for the starting point of the analysis. The first category which seems to be the most popular, considers the initial stress-free state as reference. Methods in this category include iterative finite element method variations,

beam analogy on the cables, as well as the use of nonlinear constitutive equations to mathematically denote slackening. In order to obtain the solution Buchholdt and McMillan [4] and Buchholdt [5] presented several minimization techniques such as minimization of the total potential energy. Mitsugi and Yasaga [6] presented a simple scheme for cable analyses in which the exact axial strain and a nonlinear constitutive equation of cable elements were used. The scheme turned out to be quite useful because the initial cable length can be specified exactly for the actual hardware fabrications. This is succeeded by requiring the cable lengths and tensions in the initial stress-free state as input for the analyses. Similar methods are employed by Yamamoto [7] and by Miura and Miyazaki [8]. These methods however, show some problems when detailed and accurate analyses are required. Mitsugi [8] presents a nonlinear static cable analysis method that avoids all the above problems. In his paper the equilibrium equation which virtually provides the unbalanced force at the designated variables and the stiffness matrix at the current configuration are derived for the Newton iteration. Cables under compressive strain are canceled at the integration for the global equilibrium to account for the effect of slackening. However, creep and hysteresis-type material nonlinearities are not included in the formulation presented. Krishna [9] has used the Newton-Raphson method and its variations, such as reducing load increments, to solve the nonlinear stiffness equations. Herbert and Bachtell [10] introduced the Green-Langrange strain of a cable and solved for the tension state in a cable network in conjunction with the displacements fields using Newton's method of iteration. The method successfully eliminates rigid body motions of the cable elements but the effects of the higher order strains and cable slackening are not explicitly mentioned.

When the analysis assumes that the initial geometry and tension in a cable or cable network is known, it is difficult to estimate the initial tensions in the cables when they constitute a complicated network, which is a typical situation. When a structural analysis

procedure is used by specifying a shape at the loading free state, an iterative process is necessary to obtain the desired shape under the major loading. Starting the analysis from a shape at the loading-free state, however, risks some elements becoming slack under the major loading. On the other hand, the use of slack elements at the loading-free state, which could be preferable from the viewpoint of the structural performance, is difficult. Because the emphasis is on structural performance and the shape under the major loading, it would be preferable to analyze the cable networks for a proposed shape under the major loading, then determine the required cross sectional areas and stress-free lengths of the elements. Because of this we have the work which falls in the second category of analysis of cable structures which considers as reference the shape under the application of the major loading. Schek [11] obtained this by converting the analysis problems into nonlinear programming problems. The difference between the proposed shape by an architectural viewpoint and the equilibrium shape under the major loading, and the difference between states of real tensions and prescribed tensions by a designer for some of the elements are optimized. Optimization is achieved by treating force densities, defined by internal tension of each element divided by its length, as independent variables. The coordinates of the movable nodes are eliminated from the optimization problem by substituting the force densities in the equilibrium equations. The formulation is efficient when determination of a shape close to the proposed shape under the major loading is the sole target of the optimization problem. Nishino et al. [12] present the design analysis of cable networks as nonlinear programming problems with a systematic classification of networks. A discrete model approach, by lumping the self weight into concentrated loads at nodes, is proposed with multicriteria optimization under multiple loading conditions.

Most of the literature found in this survey deals with the analysis of cable networks alone. A lot of work has been found in the literature that deals with analysis of

deployable antennas for space structures and communication satellites. Very little work has been found concerning the analysis of FC structures. The author believes that this is attributed to the fact that most of the frame structures which support the cable networks are very stiff; therefore the cable network is considered to be supported at fixed points. However, when the frame supports are flexible or when the tensions in the cables are very large, which is likely to be the case in large span structures, the assumption of fixed base nodes falters. In such cases the behavior of the frame support itself and its elastic deformation has to be taken into account. This is very significant because the structural integrity of a cable network greatly depends on the supporting structure even after the post-fabrication surface adjustment has been made.

As previously mentioned, most of the work related to structures that incorporate cables can be categorized in two major categories. It was also pointed out that the analysis of FC structures in particular consists of shape finding and loading analysis. It seems, from the literature review, that the work in the second category is related to the shape finding and several methods are presented for shape or form finding. Also, most of the work in the first category seems to be related to the classical methods, and several iterative variations of the direct stiffness method or the more general finite element analysis methods are presented. It has been realized that the work related to such structures is separated in the two categories because none of the two approaches can overpower the other. It is therefore logical to assume that both categories have advantages and disadvantages and a good solution may be found if the two approaches are compromised.

It has been conceived that the Force Density Method (FDM) [11] which is a form finding method and the Direct Stiffness Method (DSM) [13] may be integrated. The resulting technique is a combination of the two methods which are executed successively until the solution is obtained. Essentially the technique requires the FDM to obtain a

shape for the cable part of the structure. The cables are modeled as pin connected rods supported at fixed points (nodes). The linear DSM is then employed to account for the deformations in the frame structure which were not taken into account. The two methods are executed successively, each starting from the result of the other until they converge to a solution. The implementation of the FDM, the DSM as well as their final integration and convergence techniques is the work presented in this dissertation.

CHAPTER 2
FORCE DENSITY METHOD

2.1 Introduction

The analysis of cable networks has never been an easy task. Since the cable elements are much more flexible than any other type of elements, predefined cable shapes used to be analyzed nonlinearly. This, however, involved a large number of iterations and it was time consuming, as well as numerically inefficient. In addition to those drawbacks, the initial shape of the network was rarely known except in very simple cases. Consider the simple cable network of Figure 2.1. The network consists of three cable elements, two movable nodes (node 1, node 2) and two fixed nodes (node 3, node 4). The loading is applied on the movable nodes shown in the Figure 2.1 below.

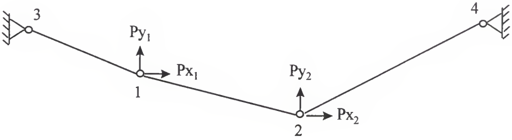

Figure 2. 1 Simple Cable Network

The equations below are necessary to balance the external loading with the internal tensions and keep the structure in equilibrium. Note that the sign in the equilibrium equations is taken care from the differences in the cable end coordinates (Δx, Δy).

$$\frac{\Delta x_1}{L_1}S_1 \pm \frac{\Delta x_2}{L_2}S_2 = Px_1 \tag{2.1}$$

$$\frac{\Delta y_1}{L_1}S_1 \pm \frac{\Delta y_2}{L_2}S_2 = Py_1 \tag{2.2}$$

$$\frac{\Delta x_2}{L_2}S_2 \pm \frac{\Delta x_3}{L_3}S_3 = Px_2 \tag{2.3}$$

$$\frac{\Delta y_2}{L_2}S_2 \pm \frac{\Delta y_3}{L_3}S_3 = Py_2 \tag{2.4}$$

where:

i = Cable number.
Δx_i = Difference in X-coordinates for each cable.
Δy_i = Difference in Y-coordinates for each cable.
L_i = Current cable length.
S_i = Current cable tension.
j = Cable node number.
Px_j = Applied load in X-direction.
Py_j = Applied load in Y-direction.

If the shape of the cable network is predefined, then the structure in Figure 2.1 is over determined to the first degree and an iterative technique is required to solve the equations for the unknown applied tensions in the cables. On the other hand, if the shape of the network is not predefined, then the number of the unknowns is greater than the number of equations and the system cannot be solved unless other simplifying assumptions are taken into account. Schek [11] managed to transform the analysis of general networks into a linear problem. For that he only considered that the final network shape should be in an equilibrium state. The linearization is accomplished by assigning force-to-length ratios to each member of the network. These force-to-length ratios proved to be good parameters for the description of the network and they are referred to as the Force Densities (FD). Any equilibrium state can be obtained by solving a linear set of equations which are expressed as the functions of the FD and the external loading. The solution that is provided by the method consists of the final coordinates of the free nodes

and a set of tensions in the cables that are required to maintain the cable network in equilibrium.

The Force Density Method (FDM) can be extended to nonlinear problems with the implementation of external constraints. The linear as well as the nonlinear FDM are presented in subsequent sections.

2.2 The Linear Force Density Method (FDM) [11]

In order to keep the two-dimensional structure of Figure 2.1 in equilibrium, the aforementioned equations have to be satisfied. The system of equations consists of four unknowns which are the coordinates (x, y) of the movable nodes (Note that the FD ratios S_i / L_i are assigned for each cable). The system of equations is determinate (linear) since there are four equations and thus it can be solved for the unknown coordinates of the movable nodes. The FDM essentially solves similar equations for each cable network under consideration. Obviously the number of equations increases for large cable networks. The FDM is a matrix method since the use of matrices for the solution of large systems of equations is essential. The following sections provide a description for the basic matrices that are used as well as the equilibrium equations which are expressed in terms of these matrices [11].

Branch node matrix $[C, C_f]$. The first step in the force density method is to define the network connectivity. For this purpose the branch node matrix, as defined by J.H. Argyris [14] and S.J. Fenves and F.H. Brannin [15], is incorporated. The branch node matrix consists of m-number of rows, equal to the number of branches (cable elements) in the network and n-number of columns, equal to the number of nodes. The matrix is subdivided into two regions. The C part of the matrix represents the columns corresponding to the free nodes and the C_f part which represents the columns which correspond to the fixed nodes. The branch node matrix is a graph of the cable network

denoting which branches are connected to which nodes. Therefore, for branch i, spanning between j and k, the entries of the branch node matrix are as follows:

$$c_{(i,\, j\ or\ k)} = \left\langle \begin{array}{ll} +1 & for \quad j_{(i)} \\ -1 & for \quad k_{(i)} \\ 0 & for\ other\ nodes \end{array} \right\rangle \tag{2.5}$$

where:

i = Indicate the row number.
j or k = Indicate the column number.

The branch node matrix for the structure of Figure 2.1 is shown below:

$$[C, C_f] = \begin{bmatrix} 1 & 0 & -1 & 0 \\ 1 & -1 & 0 & 0 \\ 0 & 1 & 0 & -1 \end{bmatrix} \tag{2.6}$$

When the connectivity of the cable network is defined, the projections of each element on the coordinate axes (x, y, z) can be expressed in terms of the branch node matrix as shown in Equation 2.7 below:

$$\begin{aligned} \Delta x &= u = Cx_r + C_f x_f \\ \Delta y &= v = Cy_r + C_f y_f \\ \Delta z &= w = Cz_r + C_f z_f \end{aligned} \tag{2.7}$$

where:

x_r = Vector that contains the released nodes.
x_f = Vector that contains the fixed nodes.
x = $\langle x_r, x_f \rangle$.

The equilibrium equations 3.1 - 3.4 can also be expressed in matrix form as shown in Equation 2.8:

$$C^t U L^{-1} s = px$$
$$C^t V L^{-1} s = py \tag{2.8}$$
$$C^t W L^{-1} s = pz$$

where:

U, V, W = Diagonal matrices which contain the vectors *u, v, w* respectively.
L^{-1} = Diagonal matrix which contains the lengths of the cable elements.
s = Vector which contains the tensions in the cable elements.
p_x, p_y, p_z = Vectors with the applied nodal loads.

Substituting $L^{-1} s = q$, $Uq = Qu$, $Vq = Qv$, $Wq = Qw$ and the element projections (*u, v, w*) on the axes in the above equations, the equilibrium equations can be further modified and take their final form which is shown in Equation 2.10 below:

$$C^t Q C x + C^t Q C_f x_f = px$$
$$C^t Q C y + C^t Q C_f y_f = py \tag{2.10}$$
$$C^t Q C z + C^t Q C_f z_f = pz$$

where:

Q = Diagonal matrix which contains the FD of the cable elements.
x, y, z = Vector which contains the coordinates of the movable nodes.
x_f, y_f, z_f = Vectors which contain the coordinates of the fixed nodes.

Force density matrix (*D*). The product $C^t Q C$ is a square matrix which is defined as the force density matrix. The force density matrix has *l*-number of rows and columns which is equal to the movable nodes. It turns out that the *D* matrix can be assembled directly instead of multiplying the individual matrices. Its entries consist of the FD of the branches connected to the movable nodes and its form depends on the branch node matrix which is defined above. The entry $D_{(i,j)}$ on the diagonal position is equal to the sum of the force densities of the branches that are framing into node *i*; the off-diagonal term $D_{(i,k)}$ is equal to the negative value of the force density of the branch connecting the nodes *i* and *k*. The *D* matrix plays analogous role in the force density method as the stiffness matrix plays in the direct stiffness method. In fact it is actually equivalent to the linear geometric stiffness matrix for the given structure. Therefore, it is not surprising that the two

matrices share the same properties; they are both symmetric and positive definite. In the same manner the two matrices can be assembled in a similar way. The D matrix of the cable network in Figure 2.1 is shown below:

$$D = \begin{bmatrix} q_1 + q_2 & -q_2 \\ -q_2 & q_2 + q_3 \end{bmatrix} \tag{2.11}$$

Fixed force density matrix (D_f). The product $C^t Q C_f$ is defined as the fixed force density matrix. Its entries consist of the negative values of the force densities of the branches that connect the movable node in the corresponding row to the fixed node in the corresponding column. For example, the term $D_{f(i,j)}$ equals the negative value of the force density of the branch that connects the movable node i, to the fixed node j. The D_f matrix has l-number of rows equal to the number of movable nodes and m-number of columns equal to the number of fixed nodes. The D_f of the cable network in Figure 2.1 is shown below:

$$D_f = \begin{bmatrix} -q_1 & 0 \\ 0 & -q_3 \end{bmatrix}$$

Once the form of the structure (fixed and free nodes) is defined, an equilibrium shape can be obtained for a given set of prestress forces in each branch and a given set of point loads applied to the movable (free) nodes. The equilibrium equations (2.10) are solved for the unknown vectors x, y, z as shown in Equation 2.12 below:

$$\begin{aligned} x &= D^{-1}(px_r - D_f x_f) \\ y &= D^{-1}(py_r - D_f y_f) \\ z &= D^{-1}(pz_r - D_f z_f) \end{aligned} \tag{2.12}$$

The final lengths and forces in each branch are calculated from the final shape of the cable network using $L = \sqrt{\Delta x^2 + \Delta y^2 + \Delta z^2}$. The element forces can be calculated

using the set of original force densities and the equation $s = Lq$. Once the equilibrium state of the network is obtained, the original unstrained length of each member can be determined using the principles of elasticity. These are the original lengths of the branches to be used during the construction period.

2.3 Advantages and Disadvantages of the Force Density Method

The main advantage of this method is that any equilibrium shape can be obtained by solving a linear system of equations and consequently the numerical calculations that are involved are reduced to a minimum. The main disadvantage of the method is that it cannot treat multiple loads such as wind and earthquake; therefore, in the presence of multiple loads the force density method falters.

2.4 Nonlinear (Extended) Force Density Method

Multiple equilibrium shapes can be obtained using the force density method the way it is presented so far. However, there are cases where an arbitrary equilibrium state may not be satisfactory if the obtained shape corresponds to a structure which cannot serve its intended purpose. In addition to that, there are cases where a certain shape is predefined. Therefore, it is very important for one to be able to "force" the FDM to allow for external constraints, so that a suitable equilibrium shape would be obtained. Schek [11] presents, "The Extended Force Density Method". This method is an extension of the linear force density method and allows for external constraints. The external constraints could be anything that can be expressed as functions of the force densities. In addition to that, since the coordinates can be expressed as functions of the force densities, any external condition which is expressed in terms of the coordinates, can essentially be expressed as a function of the force densities. The presence of additional conditions does, however, complicate the problem. The additional conditions call for an iterative

procedure. The extended force density method is nonlinear and the degree of nonlinearity is equal to the number of additional conditions. The extended force density method is implemented the same way as any other nonlinear method. The set of linear equations is solved and the external constraints are checked whether they are satisfied. If they are not satisfied then the force densities are updated and the next iteration is executed. The force densities are updated using:

$$q^{n+1} = q^n + \Delta q \tag{2.13}$$

where:

q^{n+1} = Force density of the next iteration.
q^n = Force density of current iteration.
Δq = Incremental force density.

External Constraint Function (g). The external constraint function g, has the form:

$$g = g(x,y,z,q) = 0 \tag{2.14}$$

and it represents additional external conditions (constraints). As aforementioned the constraints are expressed as functions of the coordinates and essentially as functions of the force densities. Thus, the external constraint function has the form:

$$g^* = g(x(q),y(q),z(q),q) = 0 \tag{2.15}$$

Jacobian matrix (G^t). In order to express the coordinates as functions of the force densities, the Jacobian matrix is utilized which is an application of the chain rule. Therefore, the Jacobian matrix can be viewed as a characteristic description of each additional constraint and it can be obtained as follows:

$$G^t = \frac{\partial g^*}{\partial q} = \frac{\partial g}{\partial x}\frac{\partial x}{\partial q} + \frac{\partial g}{\partial y}\frac{\partial y}{\partial q} + \frac{\partial g}{\partial z}\frac{\partial z}{\partial q} + \frac{\partial g}{\partial q} \tag{2.16}$$

where:

$$\frac{\partial x}{\partial q} = -D^{-1}C^t U .$$

$$\frac{\partial y}{\partial q} = -D^{-1}C^{t}V \ .$$
$$\frac{\partial z}{\partial q} = -D^{-1}C^{t}W \ .$$

When the external constraints and the Jacobian matrix are defined as described in the previous sections, Δq can be calculated using:

$$G^{t}\Delta q = r \tag{2.17}$$

where:

r = Vector that holds the unbalanced external constraints.

Obviously, in many cases the number of the external constraints is less than the number of branches in the network. That is, not all the branches are constrained. In such cases the system of equations is over determinate and a unique solution does not exist. In order to obtain the best solution for the system, the least squares method and its variations are implemented and the following procedure is followed to calculate the incremental force density, Δq.

$$\begin{aligned} r &= -g^{*}(q^{(0)}) \\ T &= G^{t}G \\ k &= T^{-1}r \\ \Delta q &= Gk \end{aligned} \tag{2.18}$$

Once the incremental force density is obtained, the force densities are updated and the next iteration of the nonlinear procedure is performed, with the updated values of the force densities substituted in the set of linear equations. It is obvious that when the external constraint function g, and the Jacobian matrix G^{t} are defined, the implementation of an external constraint is quite simple. In the following sections we provide the external constraint functions and Jacobian matrices that are imposed on the force density method during the analysis of the structures that we consider in this work.

2.4.1 Node Distance Constraint

Suppose that the length of the first r branches of the network have predefined values, that is the lengths of each branch (cable) has a specific value. This assumption is very common when the stiffness of the branches is very large. Therefore, the solution from the force density method has to account for the node distance conditions or the length of the specified branches. The following external constraint function and Jacobian matrix are used:

$$g = \bar{l} - \bar{l}_d = 0 \tag{2.19}$$

where:

$\bar{l}$ = Vector that holds the required lengths of the constraint branches.
$\bar{l}_d$ = Vector that holds the current lengths of the constraint branches.

2.4.2 Unstrained Length Constraint

The equilibrium shape that is obtained from the force density method is based on the equilibrium of forces at each node. The original lengths of the branches can be back calculated based on the principles of elasticity. Thus, the material properties of each branch come into play only after the final shape is obtained. In order to introduce the material properties of each element in the analysis, the unstrained length constraint is required. This constraint accounts for the strains in the members. The following external constraint function and Jacobian matrix are used:

$$g = \bar{l} - \bar{l}_u = 0 \tag{2.20}$$

$$G_u^t = -\overline{L}_u^2 \overline{H}^{-1} - \overline{L}_u^2 \overline{L}^3 (\overline{U}\overline{C}D^{-1}C^t U + \overline{V}\overline{C}D^{-1}C^t V + \overline{W}\overline{C}D^{-1}C^t W) \tag{2.21}$$

where:

$\bar{l}$ = Vector that contains the required unstrained lengths of the constraint branches.
$\bar{l}_u$ = Vector that contains the current unstrained lengths of the constraint branches.

$\overline{L}_u$ = Diagonal matrix. Its entries are the values of $\bar{l}_u$.
H = Diagonal matrix. It contains the stiffness of each branch ($h = A*E$).
Note: All matrices with the overbar denote the parts of the original matrices which correspond to the constraint branches of the network.

In each iteration a new set of coordinates is obtained which result in new length for each branch. Based on these lengths, the current branch tensions and the branch material properties, the current unstrained length l_{uj} for each branch, is calculated. The original unstrained lengths l_j, have known values. This constraint ensures that the initial unstrained length and that from the final solution are equal. The unstrained length l_{uj}, for each branch j, in each iteration is calculated as follows:

$$l_{uj} = \frac{h_j}{h_j + S_j} l_j \tag{2.22}$$

The force density method and some of the external constraints that are implemented in this work have been described in this chapter. Obviously the subject "Force Density Method" is much broader than the scope of this work. The discussion in this chapter aims to introduce the unfamiliar reader to the Force Density Method. The use of the force density method for the analysis of the structures under consideration is provided in the relevant chapters which describe the different structural systems in detail.

CHAPTER 3
CABLE STATICS AND IMPLEMENTATION OF A CABLE ELEMENT IN THE FORCE DENSITY METHOD

3.1 Introduction

Cable structures respond in a nonlinear fashion to both prestressing and in-service forces. Pre-stressing forces are those which exist in a static equilibrium configuration of the structure subjected to static load only. They stabilize the structure and provide stiffness against further load. In-service forces on the other hand are those variable live loads which the structure may be expected to encounter during its service life. In this work the analysis of cable structures is obtained with the aid of the Force Density Method. The concept of the Force Density Method is presented in Chapter 2. During the application of the Force Density Method the different branches are modeled as linear elements and the analysis is obtained from the solution of the equilibrium equations. The weight of each element is lumped at the element's end points. The result of the Force Density Method consists of the equilibrium shape of the network and the set of tensions required to maintain the network in equilibrium.

The use of linear elements which are used in the Force Density Method implies that the self weight along the cable arc is neglected. Therefore, the linear elements do not explicitly model the true cable behavior. Rather, they model truss behavior. Depending on the nature of the load (especially the live load), it is possible that some of the truss elements may be in compression. Trusses can support compression; however, this is not true for cable elements. In the case of cables, the analysis obtained from the Force Density Method is not correct. In reality, the elements (cables) which are shown to be in

compression become slack; which raises the question: "What happens when a cable becomes slack?" If the cable element is considered to be weightless then the element force is zero. However, this is not true. Leonard [16] states: "The deflected geometry of a cable however, is sensitive to load patterns and magnitudes. It is therefore, essential to consider the distributed load along the cable arc" (pg. 35).

The linear element which is used in the Force Density Method is modified to model cable behavior. The modified element is not able to support any compressive force. The tensile force is a function of the cable element weight which is uniformly distributed along the cable arc, and the cable span. This chapter presents the modified cable that is used in the Force Density Method. The classic catenary solution for the analysis of a cable element subjected to uniform load along the cable arc is discussed here. The tensile force in the cable element due to the cable weight is obtained in this manner. The chapter also presents how the result of the catenary solution is modified so the true cable behavior is modeled in the Force Density.

3.2 Uniform Load Applied Over a Catenary Segment

When a cable supports a load that is uniform per unit length of the cable itself, such as its own weight, it takes the form of a catenary. This section presents the classic catenary solution of a cable element subjected to uniform load along the cable arc [16]. The different cable parameters i.e. tension etc., are expressed in terms of the horizontal component of the cable tension, H_o. Thus, all the cable related parameters can be calculated when the horizontal component of the tension is obtained. The material is considered to be linearly elastic. The cable itself is assumed to be perfectly flexible so that the bending moment at any point of the cable must be zero.

The catenary solution which is presented below refers to Figure 3.1 which shows a typical cable element. The cable element is supported at the cable ends which do not necessarily have to be on the same level. The horizontal span is L. The load on the cable consists of its own weight which is uniformly distributed along the cable arc. Also shown on Figure 3.1 is the local coordinate system of the cable and the cable related parameters.

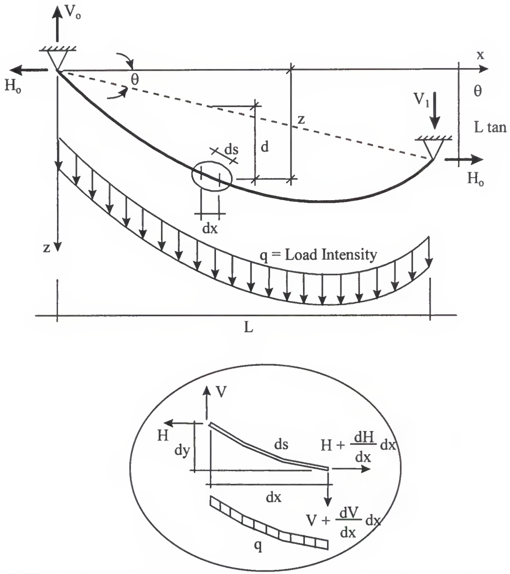

Figure 3. 1 Typical Cable Element Subjected to Uniform Weight Along its Arc

Since the applied loads are vertical, the horizontal component of the cable tension must be constant throughout the cable element. Therefore, for the differential length dx in Figure 3.1:

$$\frac{dH}{dx} = 0 \rightarrow H = H_o \tag{3.1}$$

Also, from the equilibrium of the vertical forces on the same differential length:

$$\frac{dV}{dx} = -q\frac{ds}{dx} \tag{3.2}$$

where:

q = Uniform load along the cable arc.

The cable tension must be directed along the tangent of the arc which results to the following relation between the horizontal and the vertical forces:

$$V = H_o \frac{dz}{dx} \tag{3.3}$$

The next step is to obtain the catenary equation of the cable profile. The derivative of the equation above yields:

$$\frac{dV}{dx} = \frac{d}{dx}\left(H_o \frac{dz}{dx}\right) = H_o \frac{d^2z}{dx^2} \tag{3.4}$$

Also from the differential length in Figure 3.1 $ds = \sqrt{dx^2 + dz^2}$ which yields:

$$ds = \left(\sqrt{1 + \left(\frac{dz}{dx}\right)^2}\right)dx \tag{3.5}$$

Finally the catenary equation is obtained by substituding equations 3.4 and 3.5 in equation 3.2. The catenary equation for the deflecting cable profile is shown below:

$$\frac{d^2z}{dx^2} + \frac{q}{H_o}\sqrt{1 + \left(\frac{dz}{dx}\right)^2} = 0 \tag{3.6}$$

The solution of the above equation can be obtained by integrating it twice and applying the boundary conditions $z = 0$ at $x = 0$ and $z = L\tan\theta$ at $x = L$. The solution is shown below:

$$z = \frac{H_o}{q}\left\{\cosh(\gamma + \beta) - \cosh\left[\gamma + \beta\left(1 - 2\frac{x}{L}\right)\right]\right\} \tag{3.7}$$

where:

$\beta = \dfrac{qL}{2H_o}$.

$\gamma = \sinh^{-1}\left(\tan\theta \dfrac{\beta}{\sinh\beta}\right)$.

When the catenary equation is obtained, all the cable related parameters can be calculated by direct substitution. The cable tension can be calculated as follows:

$$T = H_o\sqrt{1 + \left(\frac{dz}{dx}\right)^2} = H_o\cosh\left[\gamma + \beta\left(1 - 2\frac{x}{L}\right)\right] \tag{3.8}$$

The sag ratio at the midspan is:

$$f = \frac{1}{2\beta}\left[\cosh(\gamma + \beta) - \cosh\gamma\right] - \frac{1}{2}\tan\theta \tag{3.9}$$

Also the vertical component of the force is obtained from:

$$V = H_o\sinh\left[\gamma + \beta\left(1 - 2\frac{x}{L}\right)\right] \tag{3.10}$$

The cable stretched length can be calculated from:

$$\frac{S}{L} = \frac{V_o - V_1}{qL} \tag{3.11}$$

where :

V_o = Vertical force at $x = 0$.
V_1 = Vertical force at $x = L$.

Finally, the unstretched cable length, S_o, is determined from the differential equation:

$$\frac{ds_o}{dx} = \frac{T/H_o}{1 + T/AE} \approx \left(\frac{T}{H_o}\right)\left[1 - \frac{H_o}{AE}\left(\frac{T}{H_o}\right)\right] \tag{3.12}$$

where:

AE = Cable Stiffness.

The solution of the equation above can be obtained by integration. The boundary conditions $s_o = 0$ at $x = 0$ and $s_o = S_o$ at $x = L$ are applied.

$$\frac{S_o}{L} = \frac{V_o - V_1}{qL} - \frac{qL}{2AE}\left[\frac{H_o}{qL} + \frac{V_o T_o - V_1 T_1}{(qL)^2}\right] \tag{3.13}$$

As previously stated, all the cable related parameters are expressed in terms of the horizontal component of the cable tension, H_o. Based on the current circumstances, H_o can be determined in one of two possible ways. If the sag ratio at the cable mid-span is specified, the nonlinear Equation 3.9 can be solved for β and essentially for H_o. On the other hand, if the total unstreched length S_o is specified, Equation 3.13 is solved numerically for H_o.

3.3 Use Catenary Solution to Incorporate Force Densities

The classic catenary solution was presented in the previous section. The next step is to modify the solution above so that it can be applied in the Force Density Method. This section presents the modified cable equations as implemented in the Force Density Method. The Force Density Method requires the tension in each cable element which is assumed to be constant along the element length. The cable tension that is used in this work is taken as the tension at the starting node of the cable i.e. the tension at $x = 0$. Therefore, from Equation 3.8:

$$T = H_o \cosh(\gamma + \beta) \tag{3.14}$$

In addition to that, from Chapter 2 of this dissertation:

$$T = FD * S \tag{3.15}$$

where:

FD = Force Density.

Furthermore, if both sides of Equation 3.11 are multiplied by *FD* then a relation can be obtained between the tension in the cable (which is assumed to be constant along the element lenth) and the Force Densities. Therefore, Equation 3.11 becomes:

$$FD * S = FD\frac{V_o - V_1}{q} \qquad (3.16)$$

The relation between the tension and the Force Densities can be obtained from Equation 3.15 and Equation 3.16 and it is as follows.

$$T = FD\frac{V_o - V_1}{q} \qquad (3.17)$$

Note that Equation 3.17 is also expressed in terms of the yet unknown H_o which is the horizontal component of the cable tension. In order to calculate H_o, the tension from Equation 3.14 is substituted in Equation 3.17. Equation 3.17 is rearranged to yield the equation below:

$$FD\left(\frac{V_o - V_1}{q}\right) - H_o \cosh(\gamma + \beta) = 0 \qquad (3.18)$$

Equation 3.18 can be solved numerically for H_o. When H_o is calculated, then all of the cable parameters can be obtained by direct sustitution in the equations that are presented in the previous section. Recall from Chapter 2 of this dissertation that the Force Density Method is implemented with the unstrained length external constraint. The unstrained cable length S_o, which is the unstrained length that corresponds to the particular force density, is calculated from Equation 3.13. Also the current tension in the cable can be calculated from Equation 3.14.

The cable element as shown above is two-dimensional with respect to its local coordinate system. The implemented cable element in the Force Density Method is also two-dimensional even if the cable network that the element belongs to is in three dimensions. Therefore, the local coordinate system is required to be obtained for each cable element and all the cable parameters are expressed in this system.

CHAPTER 4
DIRECT STIFFNESS METHOD - NONLINEAR ANALYSIS

4.1 Introduction

The matrix analysis of structures is a vital subject to any structural analyst. It provides a comprehensive approach to the analysis of a wide variety of structural types, and therefore offers a major advantage over traditional methods (*i.e.* moment distribution) which often differ for each type of structure. The matrix approach also provides an efficient means of describing various steps in the analysis and it is easily programmed for digital computers. Use of matrices is natural when performing calculations with a digital computer, because matrices permit large groups of numbers to be manipulated in a simple and effective manner. The stiffness method is the most common method used in matrix analysis of structures. When the analytical model of a structure is defined (geometry, support conditions, loading), no further engineering decisions are required to carry out the analysis which can be obtained by straight matrix manipulation. The Linear Direct Stiffness Method (DSM) and the Non-Linear Direct Stiffness Method (NLDSM) are presented in this chapter.

4.2 Direct Stiffness Method (DSM)

In order to illustrate the concepts of the direct stiffness method in their simplest form, consider an elastic spring as in Figure 4.1. The spring is associated with a spring stiffness, K, which is constant and relates the applied force, F, in the spring with the spring elongation Δ. When Δ is obtained, the force in the spring can be calculated based on the spring constant.

Figure 4. 1 Simple spring with stiffness K

The DSM relies on the same idea. The stiffness of an element and essentially the stiffness of the structure is similar to the spring stiffness and it is constant. When the nodal displacements of the structure that are caused from the applied loading are obtained, the forces in the element/structure can be calculated. The difference between a simple spring and a complicated structure is the number of degrees of freedom or simply stated, the number of independent displacements. The spring element can only elongate; thus, it displaces only in one direction (axially), and its stiffness is a constant number. A three-dimensional (3-D) frame element, on the other hand, can displace in six different directions at each end (axial, shear and rotation). The stiffness of a 3-D frame element and similarly the global structural stiffness is an array of elements (matrix) instead of a simple constant like the spring element. The basic principle, however, remains the same. Instead of a constant relating the spring force to the spring elongation, there is a matrix that relates the vector of applied forces to their corresponding displacements. The global structural stiffness (stiffness of the whole structure) is obtained by superimposing the stiffness of the individual elements that compose the structure. The equation is shown below:

$$K = \sum_{i=1}^{ne} T_i^T k_i T_i \tag{4.1}$$

where:

K = Global structural stiffness.
T_i = Transformation matrix that relates the element to the structure coordinate system.
k_i = Element stiffness matrix in local coordinate system.
ne = Number of elements that compose the structure.

The objective of the DSM is to calculate the nodal displacements that correspond to the external load. Then, based on these displacements, calculate the forces in the

individual elements. The following formula relates the externally applied loading to its corresponding nodal displacements:

$$K\Delta = F \tag{4.2}$$

where:

Δ = Global displacement vector.
F = Externally applied load vector.

4.2.1 Description of the Basic Parameters

The transformation of an element stiffness matrix to local coordinates can be done as follows. Consider a frame (beam) element *i* which spans between nodes *j* and *k* with constant cross section A and length L. Associated with the element are the nodal displacements, *u*, and their corresponding nodal forces (Figure 4.2).

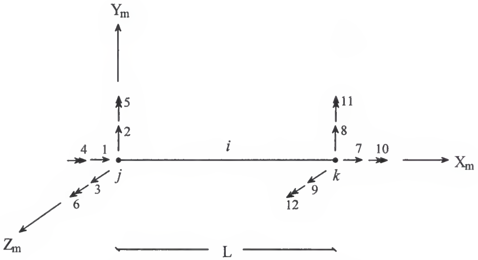

Figure 4. 2 Three Dimensional Frame DOF

The element stiffness matrix, k_i, can be generated by applying a unit value to each end displacement u_i, successively and obtaining the end moments from the slope deflection equations for the bending terms and the shear and axial forces from basic

statics and material properties. The beam element stiffness matrix k_l, is shown in Figure 4.3.

$$
k_l = \begin{bmatrix}
\frac{AE}{L} & 0 & 0 & 0 & 0 & 0 & -\frac{AE}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & \frac{6EI_z}{L^2} & 0 & \frac{-12EI_z}{L^2} & 0 & 0 & 0 & \frac{6EI_z}{L^2} \\
0 & 0 & \frac{12EI_y}{L^3} & 0 & -\frac{6EI_y}{L^2} & 0 & 0 & 0 & -\frac{12EI_y}{L^3} & 0 & -\frac{6EI_y}{L^2} & 0 \\
0 & 0 & 0 & \frac{JG}{L} & 0 & 0 & 0 & 0 & 0 & -\frac{JG}{L} & 0 & 0 \\
0 & 0 & -\frac{6EI_y}{L^2} & 0 & \frac{4EI_y}{L} & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{2EI_y}{L} & 0 \\
0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{4EI_z}{L} & 0 & -\frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{2EI_z}{L} \\
-\frac{AE}{L} & 0 & 0 & 0 & 0 & 0 & \frac{AE}{L} & 0 & 0 & 0 & 0 & 0 \\
0 & -\frac{12EI_z}{L^3} & 0 & 0 & 0 & -\frac{6EI_z}{L^2} & 0 & \frac{12EI_z}{L^3} & 0 & 0 & 0 & -\frac{6EI_z}{L^2} \\
0 & 0 & -\frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 & 0 & 0 & \frac{12EI_y}{L^3} & 0 & \frac{6EI_y}{L^2} & 0 \\
0 & 0 & 0 & -\frac{JG}{L} & 0 & 0 & 0 & 0 & 0 & \frac{JG}{L} & 0 & 0 \\
0 & 0 & -\frac{6EI_y}{L^2} & 0 & \frac{2EI_y}{L} & 0 & 0 & 0 & \frac{6EI_y}{L^2} & 0 & \frac{4EI_y}{L} & 0 \\
0 & \frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{2EI_z}{L} & 0 & -\frac{6EI_z}{L^2} & 0 & 0 & 0 & \frac{4EI_z}{L}
\end{bmatrix}
$$

Figure 4. 3 Three Dimensional Frame Stiffness in Local System [17]

Similarly consider a truss element, l, of constant cross section A and length L. The nodal displacements and forces associated with this element are provided in Figure 4.4.

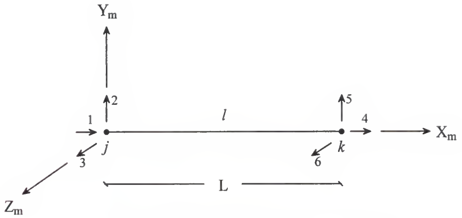

Figure 4. 4 Three Dimensional Truss Element DOF

The element stiffness matrix for a truss element is calculated in a similar manner with the beam element. The truss stiffness matrix k_l, is shown in Figure 4.5 and it can also be used for cable elements in tension. Obviously when the cables are in compression, their stiffness is zero.

$$k_l = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & -\frac{AE}{L} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ -\frac{AE}{L} & 0 & 0 & \frac{AE}{L} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix}$$

Figure 4. 5 Truss Element Stiffness Matrix [17]

Note that the stiffness matrix of a truss/cable element is smaller than the stiffness matrix of a frame. This is owed to the fact that the truss/cable elements have fewer degrees of freedom per node. These elements are not capable of providing bending stiffness and therefore the rotations are not included in the stiffness derivation. As previously mentioned, the superposition of all the element stiffness compose the global structural stiffness K.

Rotation Transformation Matrix T, of a Coordinate System. Most structures are composed of more than one element each of which may have any orientation in space. In those cases, the element stiffness which is derived in the element (local) coordinate system orientation for each element, may not be compatible with the other elements since they may not have the same orientation. Therefore, they cannot be superimposed directly. In order to be compatible, the individual element stiffness are transformed to a common coordinate system, the structural (global) coordinate system. The transformation matrix which relates two arbitrary coordinate systems is utilized to perform this task. It can be obtained with the aid of a third point that defines the local planes of the element, or it can be obtained with the aid of the Euler angles. In this work the transformation matrix is obtained with the aid of the Euler angles. The third point method is also used to setup the initial orientation of the element. The transformation matrix is presented below [17].

The element stiffness is derived based on the element degrees of freedom (EDOF) in the local coordinate system. This is the key to simplifying the assembly process. The stiffness is then transformed from the local to the global coordinate system by means of the transformation matrix. When the transformation matrix is known, the transfer of values between the local and global coordinate system is trivial. The matrix that relates the local to the global coordinate system is referred to as the rotation transformation matrix T. It is a block diagonal matrix and each block $\underline{T}$ (3x3) is identical to each other. Thus, the calculation of one block, is enough to define the whole transformation matrix.

The transformation matrix is defined in terms of the Euler angles which are shown in Figure 4.6. They define the rotations that the member undergoes about its principal axes when it is moving from one coordinate system to the other. The specific orientation of the local system is shown in Figure 4.6 also.

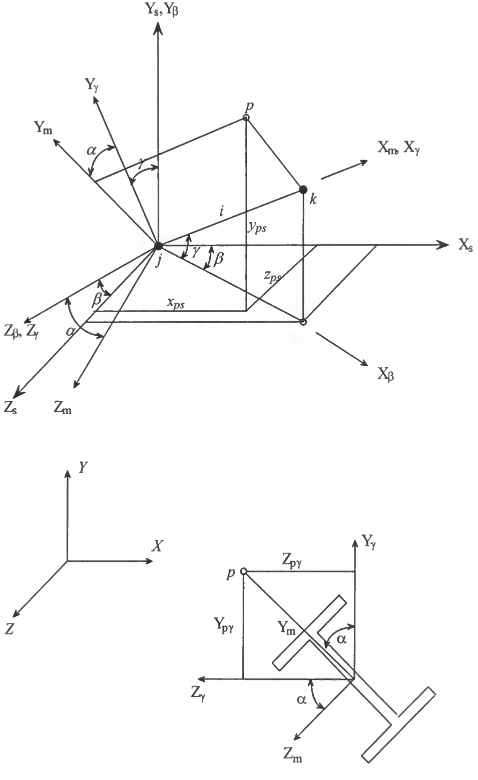

Figure 4. 6 Local Coordinate System

The x_m axis is selected as the axis along the axis of an arbitrary member, i, which spans between nodes j and k. In order to establish the other two directions the orientation of the global coordinate system has to be established. For that the axes x_s, y_s, z_s are chosen which are parallel to the axes of the global coordinate system x, y, z. While many choices are possible for the directions of the y_m and z_m axes, we set the z_m axis as being on the x_s - z_s plane. It follows that the y_m axis completes a right handed coordinate system and therefore it lies on the x_m - y_s plane. When the local axes are defined in such a way there is no ambiguity about their orientations except in the case of a vertical member. The vertical member is discussed later.

There are cases where the measurement of α (rotation about x_m) is difficult. In such cases a third point p, is introduced to describe the orientation of one of the local planes. Point p, is shown on Figure 4.6 also. In this work the third point p, is used to define the strong plane of the element (x_m, y_m). The general form of $\underline{T}$ is given below. The three elements in the first row are the direction cosines for the local x_m, axis with respect to the global x, y, z axes. The elements in the second and third rows are the direction cosines of the local y_m, and z_m, axes with respect to the global x, y, z axes.

$$\underline{T} = \begin{bmatrix} Cx & Cy & Cz \\ \dfrac{-CxCy\cos\alpha - Cz\sin\alpha}{Cxz} & Cxz\cos\alpha & \dfrac{-CyCz\cos\alpha + Cx\sin\alpha}{Cxz} \\ \dfrac{CxCy\sin\alpha - Cz\cos\alpha}{Cxz} & -Cxz\sin\alpha & \dfrac{CyCz\sin\alpha + Cx\cos\alpha}{Cxz} \end{bmatrix}$$

where:

$$Cx = \frac{x_k - x_j}{L}, Cy = \frac{y_k - y_j}{L}, Cz = \frac{z_k - z_j}{L}, Cxz = \sqrt{Cx^2 + Cz^2}.$$

$$L = \sqrt{(x_k - x_j)^2 + (y_k - y_j)^2 + (z_k - z_j)^2}.$$

$$\sin\alpha = \frac{z_{p\gamma}}{\sqrt{y_{p\gamma}^2 + z_{p\gamma}^2}}, \quad \cos\alpha = \frac{y_{p\gamma}}{\sqrt{y_{p\gamma}^2 + z_{p\gamma}^2}}.$$

$$x_{ps} = x_p - x_j, \quad y_{ps} = y_p - y_s, \quad z_{ps} = z_p - z_j.$$

$$\begin{bmatrix} x_{py} \\ y_{py} \\ z_{py} \end{bmatrix} = \begin{bmatrix} Cx & Cy & Cz \\ \dfrac{-CxCy}{Cxz} & Cxz & \dfrac{-CyCz}{Cxz} \\ \dfrac{-Cz}{Cxz} & 0 & \dfrac{Cx}{Cxz} \end{bmatrix} \begin{bmatrix} x_{ps} \\ y_{ps} \\ z_{ps} \end{bmatrix}$$

x_p, y_p, z_p = Coordinates of an arbitrary point which lies on the strong plane of the member.

In the case of a vertical member, the principal axes of the element will be rotated about the vertical axis so that they form an angle α, with the structural axes. A vertical member is shown in Figure 4.7. There is no rotation about y_s ($\beta = 0$). The rotation is through the angle γ which may be 90° or 270°. The rotation α about the element axis (x_m) is also shown on the Figure. The rotation transformation matrix can then be obtained by inspection and it is shown below. The value of Cy needs to be adjusted as +1 when $\gamma = 90°$ and -1 when $\gamma = 270°$.

$$\underline{T} = \begin{bmatrix} 0 & C_y & 0 \\ -C_y \cos\alpha & 0 & \sin\alpha \\ C_y \sin\alpha & 0 & \cos\alpha \end{bmatrix}$$

The DSM solution procedure can be found in many textbooks related to matrix analysis of structures. The solution procedure as implemented in this work is shown below [13]:

4.2.2 Direct Stiffness Method Analysis Procedure

1. Define the global DOF for the structure.
2. Find the Transformation matrix for each element in the structure.
3. Assemble the global (structure) stiffness matrix.
4. Form the externally applied load vector.
5. Solve for the nodal displacements.
6. Recover the element forces based on the nodal displacements.

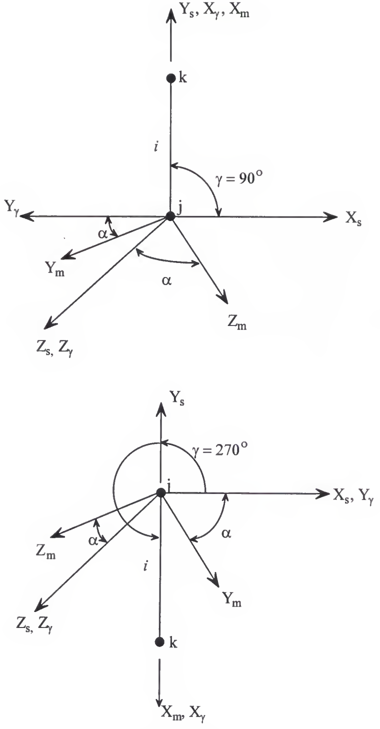

Figure 4. 7 Vertical Member Orientation in Local Axis Definition

4.3 NonLinear Direct Stiffness Method (NLDSM)

Nonlinearity of structures can be classified as material nonlinearity (associated with changes in material properties, as in plasticity) or as geometric nonlinearity (associated with changes in configuration, as in large deflections of a slender elastic beam). The classifications "linear" and "nonlinear" are artificial in that physical reality presents various problems, some of which can be satisfactorily approximated by linear equations. It is fortunate that linear approximations are quite good for many problems of stress analysis. Nonlinear approximations are more difficult to formulate, and solving the resulting equations costs much more than a linear approximation having the same number of degrees of freedom (DOF). Many physical situations present nonlinearities too large to be ignored. A change in configuration may cause loads to alter their distribution and magnitude. An analyst must understand the physical problem and must be acquainted with various solution strategies. A single strategy may not always work well, and may not work at all for some problems. Several attempts may be needed in order to obtain a satisfactory result. Nevertheless, nonlinear analyses are undertaken more often than in the past. In part this is because computing costs have declined. In addition to that, more demands are placed on the structures.

Geometric nonlinear behavior. When the displacements of the structure are large, the entire geometry of the structure changes. Thus, the moment arms in the structure change. Also the shears may not be vertical and at the same time, the axial forces may not be parallel, to the original axes. As a result, secondary forces develop (Figure 4.8) that are not accounted for and therefore the results from the DSM may be erroneous. Linear analysis assumes that the displacements are small and equilibrium can be checked in the undeformed configuration.

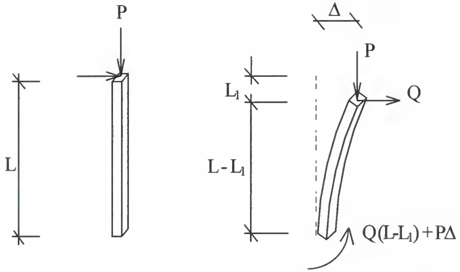

Figure 4. 8 Secondary Moments Due to Geometric Nonlinearities

Material nonlinear behavior. This type of nonlinear behavior is associated with the relation of stress and strain of the material that the structure is composed of. When the stress in the material exceeds the elastic limit the Hooke's Law (which states a linear relation between stress and strain) does not apply and the recovery of the internal forces is not trivial.

Like most techniques developed so far, the DSM was developed based on the assumption that the element deformations which take place under the application of the loading are small and that there is a linear relation between the stress and strain in the elements that compose the structure. Unfortunately there are times when the displacements are not small or the stress-strain relation of the elements is not linear. When the DSM assumptions are violated the method cannot be used directly. In such cases a nonlinear variation of the DSM has to be implemented which consists of a series of linear analyses. The nonlinear variation is an iterative technique and accounts for the different nonlinearities which are present. In the analysis of the structures under consideration in this work, it is certain that geometric nonlinearities are present. This is due to the fact that the cable elements are very flexible. Based on the large displacements

of the cable members, large strains may be induced and thus, the presence of material nonlinearities is possible. However the large yield point of the cable elements permits us to ignore the material nonlinearities in this problem. In this work a nonlinear variation of the DSM which includes geometric nonlinearities is utilized and the solution strategy is presented in the next section.

4.4 Tangent Stiffness (TSM)

The nonlinear DSM is an iterative procedure that performs a series of linear steps, until the answer converges to the solution. Convergence is achieved when the external load is balanced by the internal resisting force that is developed in the elements. This is the equilibrium state and therefore the solution. The Tangent Stiffness Method (TSM) provides a way of modifying the stiffness of the structure in each iteration as its geometry changes, so that the solution proceeds to convergence. The TSM was chosen over other methods (*i.e.* initial stiffness, secant stiffness) since convergence is obtained faster.

Each material is characterized by a stress strain curve. Based on this relation, it is possible to construct P-Δ curves (Figure 4.9) for each element.

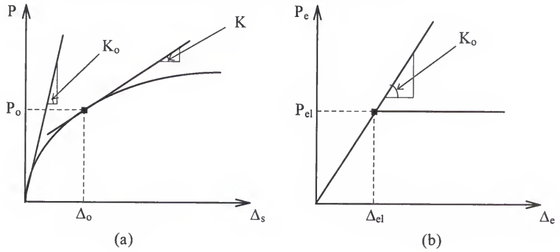

Figure 4.9 Sample P-Δ Curves for Structure and Individual Elements

The curves are graphical representations of the relation between the applied load and its corresponding deformation. That is, a certain amount of deformation that the element undergoes is associated with a corresponding load. This relation can be linear (straight line) or nonlinear (curve). The slope of the curve at each point corresponds to the stiffness of the element. Similarly P-Δ curves for an assemblage of elements (whole structure) can be constructed, the construction of which is not trivial since it involves a combination of all the elements.

Suppose the curve in Figure 4.9a represents the P-Δ curve for a structure and Figure 4.9b represents the P-Δ curve for an element. First a linear analysis is performed on the structure using the initial stiffness K_o. Based on the displacements and the element P-Δ curve (Figure 4.9b), the internal resisting force in each element is obtained. The forces for all elements are superimposed to a global resisting force which represents a point on the curve for the whole structure (Figure 4.9a). This force is then compared to the applied load. If the two forces are not equal, the stiffness of the individual elements are modified based on the current configuration and the element P-Δ curves. In this problem a linear stress-strain relation is used. The global stiffness of the structure is defined again based on the modified stiffness of the individual elements. Theoretically, the modified stiffness of the structure is equal to the slope of the structure P-Δ curve at the particular point (tangent method). The difference between the internal resisting and external force becomes the new applied load. This load is referred to as the unbalanced load since this is the load that force the structure to be out of balance. A new linear analysis is performed using the new parameters and the same process continues until the internal resisting force balances the external load, within a certain tolerance. Then the structure is in equilibrium.

Unbalanced load vector (UL). As the name suggests, this vector contains the load which is needed to balance the external with the internal force. That is, the unbalanced

load equals the applied load minus the resisting force. The solution is obtained when the unbalanced load equals zero. The best way to show how the tangent stiffness method works is through a simple but very illustrative example.

Example 4.1

The structure of Figure 4.10 is to be analyzed using the NLDSM. The applied loading of 25 N is applied on the weightless plate which is supported by linear springs as shown in the Figure 4.9. The springs are identical and their spring constant, k = 10 N/cm. It is required to find the equilibrium state of this structure as well as the force that is developed in each spring. The author uses this simple example to provide the general idea of the TSM. It should be noted, that the reader should spend some time trying to understand the concept that is understated in this example. The solution is obtained in three iterations. The intermediate steps of the procedure are tabulated in Table 4.1.

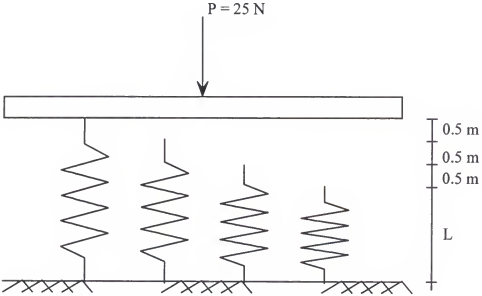

Figure 4.10 Spring Structure for the Illustrative Example

4.4.1 Nonlinear Direct Stiffness Method Analysis Procedure

1. Assume the initial structural stiffness, $K_o = \sum_{i=1}^{ne} T_o^T k_{o_i} T_o$
2. Set the structural stiffness, $K = K_o$.
3. Set the externally applied force as the unbalanced load, $UL = F$.
4. Calculate the structural displacements, $r = K^{-1}UL$.
5. Calculate the force in each spring based on r and the branch stiffness.
6. Get the total resisting force, R.
7. Calculate the imbalance load, $UL = F - R$.
8. If the UL is not zero proceed to step 8 otherwise calculate the element forces and stop.
9. Assemble the modified structural stiffness $K = \sum_{i=1}^{ne} T^T k_i T$
10. Repeat steps 4 to 8.

Table 4.1 Sample Tangent Stiffness Iteration

Iteration No.	k_a N/cm	k_b N/cm	k_c N/cm	k_d N/cm	Σk_i N/cm	$r = \frac{UL}{\Sigma k_i}$ cm	Σr cm	$S_a = k_a r_a$ N	$S_b = k_b r_a$ N	$S_c = k_c r_a$ N	$S_d = k_d r_a$ N	$R = \Sigma S_i$ N	UL = F-R N
1	10	0	0	0	10	2.5	2.5	25	20	15	10	70	-45
2	10	10	10	10	40	-1.125	1.375	13.75	8.75	3.75	0	26.25	-1.25
3	10	10	10	0	30	-0.042	1.333	13.33	8.33	3.33	0	25	0

CHAPTER 5
FRAME-CABLE STRUCTURE ANALYSIS

5.1 Introduction

A Frame-Cable (FC) structure was defined in an earlier chapter as a pre-stressed cable network which is supported by a space frame. Any FC structure can be thought of as a structure which consists of two different substructures; a cable network and a space frame. Each of the substructures is an assemblage of elements which are interconnected at nodes. A node is defined as the physical location of the structure where two or more elements are connected to each other. The cable network consists of cable elements which are connected to each other at the movable nodes. The cable network is attached to the space frame which it is supported. The common nodes of the cable network and the space frame are referred to as the cable base nodes. The space frame transfers the loading to the foundation. The common nodes of the frame elements and the foundation are referred to as frame base nodes. The loading on the cable network consists of a set of point loads which are applied on the cable movable nodes. As a result of the applied loading, the cable movable nodes displace considerably and assume new positions in space. The new locations of the movable nodes define a new geometry for the cable network. The space frame consists of a number of frame (beam) elements. The loading on the space frame consists of the tensions in the cables which are attached on the space frame at the cable base nodes. The magnitudes and directions of these tensions depend on the equilibrium state of the cable network (Figure 5.1).

The fundamental question regarding the analysis of this type of systems, is whether the displacements of the space frame should be considered. In other words can

the space frame displacements be neglected, obtain a solution for the cable network only, and still obtain an accurate analysis? In the case of a very stiff space frame which is subjected to small cable tensions, this approach results in a good approximation. However, in the case of a flexible space frame, or a cable network with large cable tensions, or both, then the space frame displacements cannot be neglected. The integrity of a FC structure depends greatly on the supporting space frame and failure to account for the space frame effects will result in erroneous results with unpleasant implications.

A different technique for the analysis of this type of structures was developed which treats the two substructures separately. This technique is presented in this chapter.

5.2 Analysis Technique Overview

The cable network substructure is a highly nonlinear system and its analysis calls for an iterative solution technique i.e. the Force Density Method (FDM). The space frame substructure on the other hand remains in the linear range and therefore, a linear solution using the Direct Stiffness Method (DSM) would suffice. If the two substructures were not coupled, they could be analyzed independently using the aforementioned methods. However, this is not possible since the behavior of the whole system is affected by the behavior of both substructures. The analysis of the structures under consideration requires a technique which treats the two substructures simultaneously, i.e. the a Non-Linear variation of the Direct Stiffness Method (NLDSM). The use of the NLDSM directly is very difficult mainly because of the nonlinearities and instabilities in the cable network. The technique that is presented herein aims to the removal of those instabilities before the application of the NLDSM. That is accomplished with the aid of a series of cycles between the FDM and the linear DSM. The use of these cycles aims to the calculation of an approximate solution which serves the analysis purpose in two parts.

1. The structure at the approximate solution state is free from the instabilities in the cable network.

2. The required number of iterations in the NLDSM application is reduced to a minimum since the approximate solution is very close to the true equilibrium solution.

The major characteristic of this technique which consists of the FDM and the DSM, is that it is iterative at different levels. The technique is broken down into four major steps:

1. Identify the two substructures, define the cable movable nodes, the cable base nodes, the frame base nodes, the connectivity of the different elements and the applied loading.

2. Analyze the cable network substructure using the Force Density Method.

3. Analyze the space frame substructure using the Direct Stiffness Method including the effects of the cable network.

4. Consider the whole system and perform a non-linear analysis using the Non-Linear Direct Stiffness Method.

In addition to the major steps above it is required to have a "transition stage" at the end of each step and before the execution of the next step. The "transition stages" are required to setup the parameters and the transfer of data from one step to the other.

5.3 Analysis Technique for Frame-Cable Structures

A detailed description of the analysis technique is presented in this section. As mentioned in the previous section the technique is broken down into four major steps. These steps are presented here together with the procedure followed in this analysis technique.

5.3.1 Major Step 1

The first major step of the analysis technique is to define the parameters of the FC structure. The required parameters include the structure nodal points and their distinction to cable movable nodes, cable base nodes and space frame base nodes. Also the first major step is used to distinguish the two substructures, their connectivity and the applied loading. The cable elements are organized to form the cable network and the frame elements the space frame. The load on the cable network consists of the weights of the cables and any additional load i.e. the load due to the applied wind. The cable weight is lumped at the end nodes of each cable. Each of the two cable end nodes is loaded with half of the cable weight. The loading on the space frame consists of the tensions in the cables that are supported by it. The tensions in the cables result from the analysis of the cable network.

5.1 Example

Consider the structure in Figure 5.1a. The nodes 7, 8, 9, 10 are the cable movable nodes. The nodes 2, 3, 5, 6 are the cable base nodes and nodes 1, 4 are the space frame base nodes. The cable network consists of the cable elements that connect to all the cable nodes as shown in Figure 5.1c. Note that the cable base nodes are shown to be fixed. The space frame consists of the frame elements as shown in Figure 5.1b. Note that in this case the cable base nodes are shown free to displace. The load on the cable network is also shown on Figure 5.1c and in this case it consists of the cable weights. The load on the space frame is shown on Figure 5.1b and it consists of the cable tensions that result from the cable network analysis.

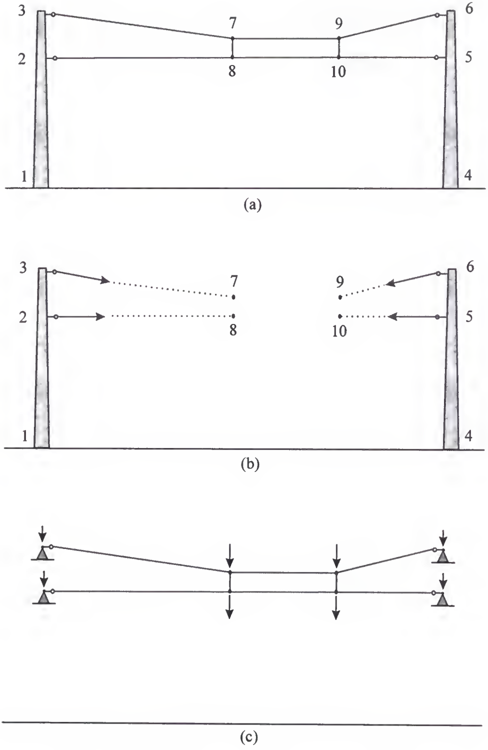

Figure 5. 1 Frame-Cable Structure, the Substructures, the Loading

5.3.2 Major Step 2

The aim of the second major step is to analyze the cable network substructure, i.e. for the structure in Figure 5.1 the cable substructure would be the structure shown on Figure 5.1c. The FDM is applied to carry out this task. The FDM as discussed in Chapter 2 and outlined here, provides the equilibrium shape for a cable network by solving a system of linear equations. The linearization is accomplished by assigning force to length ratios for each element of the network. These ratios are referred to as the Force Densities. As a result, multiple equilibrium shapes can be obtained using the linear FDM just by altering the force densities in the members. In most cases, however, a particular shape is required and the implementation of external constraints to the FDM is necessary. The resulting force density variation, the Extended Force Density Method (EFDM) is iterative and therefore, non-linear. The load is applied at the movable nodes which assume different locations in space to change the geometry of the cable network. In addition to that the tensions in the cables are adjusted accordingly to maintain equilibrium with the applied loading while satisfying the external constraints. The fixed nodes retain their locations without moving.

5.3.3 Major Step 3

The third major step deals with the analysis of the space frame using the linear DSM. This step is necessary to account for the space frame displacements which were not taken into account in the analysis of the cable network substructure in the previous step. The DSM as presented in Chapter 4 and outlined here, is a classical technique for the analysis of finite element structures. Essentially the method requires the global stiffness matrix of the system that describes a linear relationship between the displacements and the applied forces, all measured at the nodal points. The stiffness matrix is assembled

from the stiffness of the individual elements using coordinate system transformation matrices that relate the individual element coordinate systems to a global coordinate system. The loading is applied at the nodal points and it consists of the cable tensions that are obtained from the analysis of the cable network substructure in major step 2. For the structure of Figure 5.1a the space frame is the structure shown on Figure 5.1b.

5.3.4 Major Step 4

This is the last step of the analysis technique. The NLDSM is used to obtain the final equilibrium state of the structure under consideration i.e. the structure in Figure 5.1a. As discussed in earlier sections and is shown in more detail in subsequent sections the previous two major steps (major step 2 and major step 3) are used to obtain an approximate solution for the structure. The NLDSM as presented in Chapter 4 is a nonlinear variation of the DSM which accounts for large displacements. The NLDSM is used for analysis of the structure as a whole. Therefore, the two major substructures are superimposed to form the complete structure. The loading consists of the externally applied load on the structure.

5.3.5 Analysis Procedure

The different steps of this analysis technique are presented in this section. The best way to explain the different steps is to go through an example. Consider the structure shown in Figure 5.2a. The cable network substructure is shown in Figure 5.2b. The nodes 5, 6, 7, 8 are the cable movable nodes and nodes 2, 4 are the cable base nodes. The cable network connectivity as well as the applied loading is shown on the same figure. Once the cable network substructure is defined (major step 1) the FDM is employed to obtain the solution. If the network has a predefined shape (almost always), then the FDM must be implemented with the appropriate external constraints. In this case the unstrained length

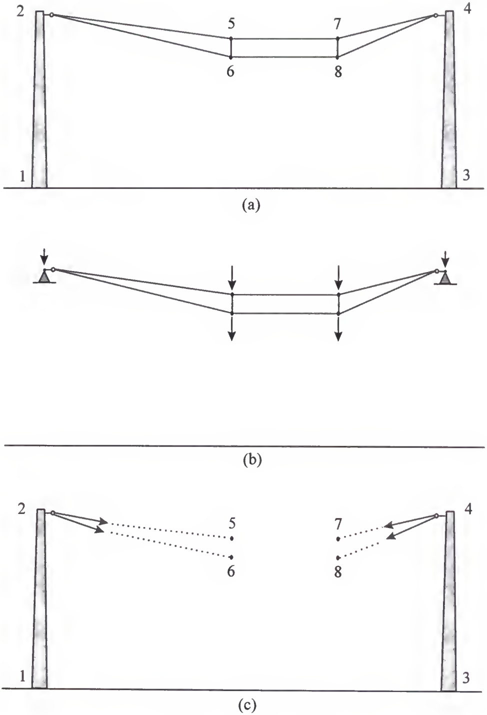

Figure 5. 2 Frame-Cable Structure

constraint as presented in Chapter 2 is implemented. The result of the FDM solution (major step 2) consists of the required network geometry (shape) and the appropriate set of tensions in the cables to balance the applied loads. Note that during the FDM analysis the cable base nodes are considered to be fixed i.e. nodes 2 and 4 in Figure 5.2b. Therefore, no displacement takes place in the space frame and as a result no stresses are developed in it. In reality however, nodes 2 and 4 displace.

When the equilibrium of the cable network is obtained the linear DSM is employed to account for the displacements in the space frame (major step 3). This step is required to obtain the displacements in the space frame which were not calculated by the FDM in the previous step; essentially this step will provide not only the displacements of the space frame but also the stresses that develop in the frame elements due to the cable tensions. The space frame substructure is shown in Figure 5.2c. The base frame nodes are considered to be fixed whereas the cable base nodes are considered to be free to displace. The loading on the on the space frame substructure is applied on the cable base nodes (node 2 and node 4) and it consists of the tensions in the cables. The magnitude of the loads are equal to the tensions in these cables. The results form the DSM consists of the new location of the cable base nodes as well as the stresses in the space frame to balance the cable tensions.

The new locations of the cable base nodes disturb the equilibrium of the cable network which was obtained by the FDM earlier (major step 2). Therefore, it is necessary to reanalyze the cable network substructure using the FDM to correct the cable network equilibrium. The new equilibrium state of the cable network, consists of a different set of tensions in the cables and consequently the space frame equilibrium state which was obtained earlier (major step 3) is not correct. Therefore, the linear DSM (major step 3) needs to be applied again to calculate the new locations of the cable base nodes and the new set of stresses in the frame elements. It is natural that a cyclic process is created with

each step adjusting the particular substructure to the equilibrium state of the previous step. The cyclic process eventually converges to a solution for the system which consists of the required shape for the cable network, a set of cable tensions, the space frame displacements and the appropriate stresses in the space frame elements to balance the loading.

When the cyclic process eventually converges to a solution, the result is actually an approximate solution for the entire system. The NLDSM is then implemented (major step 4) to obtain the final equilibrium solution and eliminate any out of balance forces that exist from the approximate result. The NLDSM is implemented as presented in Chapter 4. The method converges in a minimum number of iterations. This is owed to the fact that the approximate solution that is obtained from the cyclic process is very close to the true equilibrium solution.

In the previous section the author referred to "transition stages". It was mentioned that these stages are used for the problem setup and to the transfer of data between major steps. Actually these stages provide good places for the implementation of any convergence techniques which are used to speed up the convergence rate. Even though these stages are not presented as "major steps" in the analysis, they are equally important. As already mentioned in a previous section the implemented technique is iterative at different levels i.e. the FDM is iterative, the cyclic process FDM-DSM is iterative and finally the NLDSM is iterative as well. Different convergence techniques are implemented to aid the convergence rate of the technique at any of the different levels of non-linearity. The different convergence methods as well as the checks for the allowable tolerance are implemented during the transition stages. The different convergence methods as well as the tolerance requirements are presented in the subsequent sections.

5.4 Implemented Convergence Methods

It is apparent that the new procedure is heavily computational. In order to minimize the execution time, different convergence methods are implemented. The applied methods not only reduce the computational effort but also stabilize the procedure which in some cases tends to become numerically unstable. Convergence methods are implemented at two different stages of this procedure. The first is concerned with the convergence of the FDM solution which is nonlinear and is used repeatedly in the process. The second is concerned with the convergence of the pole displacements which are caused by the tensions in the cables as discussed in the previous section.

5.4.1 Convergence of the Force Density Method

During the execution of the FDM, each cable i, is characterized by a force density q_i which is incremented in each iteration until the solution is reached. The rate of convergence of FDM depends on the choice of the increment values of the force densities, dq_i. The incremental values are adjusted based on the Method of Adaptive Learning Rates (MALR) [18]. This method maintains a history factor $qfact_i$, for each force density which is based on the convergence pattern of the particular force density. The history factor is updated in each iteration using the following formula:

$$qfact_{i(n)} = \theta\, qfact_{i(n-1)} + (1-\theta)\, dq_{i(n-1)} \tag{5.1}$$

where:

n = Current iteration.

θ = Factor that reflects the influence of the previous iterations (history) to the current.

The values of θ range between 0 and 1. As θ approaches 1, the value of the history has more influence on the value of the current step. The process is accelerated or

decelerated if it is converging to, or diverging from the solution. MALR determines if the process is converging when $qfact_i$ times dq_i is positive. The variable dq_i is the unbalanced force density for the current iteration. The unbalanced force density is adjusted using the equation:

$$dq_{i(n)} = \begin{cases} dq_{i(n-1)} + k & \text{if } dq_n * qfact_n > 0 \\ dq_{i(n-1)} * \varphi & \text{if } dq_n * qfact_n \leq 0 \end{cases} \tag{5.2}$$

where:

k, ϕ = History parameters which range from 0 to 1 (in this case both constants equal 0.1).

If the process is heading towards convergence the step is increased by a factor k. If the direction is reversed it is scaled by a factor ϕ.

5.4.2 Convergence of the Space Frame Displacements

This section deals with the convergence of the FDM - DSM cyclic process. In this section reference will be made to cycles which refer to a complete FDM - DSM solution. For example, the end of the first FDM analysis for the cable network is followed by the first DSM solution to account for the space frame displacements and stresses. The end of the first DSM solution is also the end of the first cycle. The convergence method is deduced by the convergence pattern of the space frame displacements at individual cable base nodes. The displacements of the space frame correspond to the tensions that are obtained from the FDM solution in each iteration. The cyclic process converges when the tensions in all the cable elements of the cable network have the same magnitudes (within acceptable tolerance) between two successive cycles. In such case the displacements of the cable base nodes from the consequent DSM solution will have equal values (within acceptable tolerance) between the successive cycles.

During the cyclic process the FDM is used to obtain the solution for the current stage of the cable network. Then the DSM is used to obtain the displacements of the space frame that correspond to the tensions from the FDM. Then the next FDM is used to account for the space frame displacements and the process continues until the two methods are finally "in phase", that is the displacements of the space frame correspond to the tensions from the FDM solution. The aim of the convergence method that is presented in this section is to modify the cable tensions from the FDM solution in such a way that the consequent space frame displacements are closer to the final solution. The cable tensions are modified not only to speed up the convergence of the space frame displacements but also to ensure that the cable tensions stay within reasonable limits. The cable tensions are always modified based on reference tension values which are assigned to each cable in each cycle. The reference tension changes in every cycle. The cable tensions are modified as follows:

Cycle 1. The cable tensions from the FDM solution are applied on the space frame without any modification. These tensions however, are used in the next cycle as reference tensions. It turns out that the values of these tensions are extreme values. They are either the maximum that any cable may obtain through the cyclic process or the minimum.

Cycle 2. The cable tensions from Cycle 1 are extreme values. It also turns out that the cable tensions from the second FDM solution are the opposite extremes from those of Cycle 1. That is, if a cable tension from Cycle 1 is a maximum value then the cable tension in the same cable from the second FDM solution is a minimum. The convergence method modifies the cable tensions from the second FDM solution before they are applied on the space frame for the space frame displacements. The modified cable tensions equal the average of those obtained by the first two FDM solutions. The average tension for each cable is used as the reference tension in the next cycle.

Cycle > 2. The first two cycles are performed to set the first "reasonable" reference tension. In subsequent cycles the cable tensions are modified based on the "reference" tensions and the "current" tensions from the current FDM solution in each cable as follows:

1. If the difference between the current tension and the reference tension falls within a certain tolerance (in this case 2%) then the modified cable tension equals to the average of the two tensions. The modified tension is used as the reference tension in the next cycle.

2. If the current tension is greater than the reference tension then the reference tension is increased by a certain percentage. The modified tension is then set equal to the reference tension.

3. If the current tension is smaller than the reference tension then the reference tension is decreased by a certain percentage. The modified tension is then set equal to the reference tension.

The modified tension values are the values of the tensions to be applied on the space frame to obtain the space frame displacements. As already mentioned the cyclic process ends when the cable tensions between successive cycles fall within acceptable tolerance limits.

5.5 Tolerance

It is very important to define the accuracy of the technique. This refers to the tolerance requirements which affect greatly the efficiency of this technique. The tolerance basically measures the allowable deviation of a particular solution from the exact solution. The value that is assigned to the tolerance refers to the intention to accept an approximate solution. At intermediate stages, the analysis must be checked whether it has converged or not. In other words, at the end of each intermediate step (cycle) the current solution must be checked to assess whether it conforms with the specified tolerance limits. In such case, the process has converged to a solution and the process terminates.

The displacements of the space frame are checked at the end of each cycle and then they are compared to those of the previous cycle. The tolerance limits are set to a percentage of the change of the space frame displacements between successive cycles.

5.6 Computer Implementation

It is evident that since this technique is heavily computational, its implementation requires the use of the computer. The ATLAS software (Analysis of Traffic signal Lights And Signs) is the product of a research program which aims to the analysis and design of the traffic signal lights and signs that are used in the State of Florida. The research is sponsored by the Florida Department Of Transportation (FDOT) and it was generated after Hurricane Andrew hit South Florida and caused excessive damages in these structures.

The technique is presented with the aid of flowcharts which are shown in Figures 5.3 - 5.4. The flowchart in Figure 5.3 describes the implementation of the Force Density Method. This is referred to as the Shape Finder. Figure 5.4 shows the flowchart of the Direct Stiffness Method as well as the Non-Linear Direct Stiffness Method. The flowchart consists of two different parts. The first part describes the linear Direct Stiffness Method and is referred as the DSM solution, and the second part which shows the extra steps that are required for the linear DSM to be extended to non-linear. The second part of the flowchart is referred to as the NLDSM solution. Finally the flowchart in Figure 5.5 shows the integration of the technique that is presented in this report.

5.6.1 Shape Finder (SF)

The implementation of the FDM is shown in Figure 5.3. The program is referred to as the Shape Finder (SF) part of the procedure because it calculates the shape for the cable network given a set of loads and a set of force densities. The first step is to obtain the

input data and allocate the memory to the required data structures. The input data consists of the information about the nodal points and the information about the cable elements. The nodal point information consists of the geometry of the nodal points as well as their description i.e. fixed nodes or movable nodes. The SF abbreviates *'F'* for the fixed or base nodes and *'R'* for the released or movable nodes. The element input data consists of the element connectivity i.e. the end nodes of each element (each element is connected to two nodes only) and the element properties (Young's Modulus of Elasticity, E, and the cross sectional area, A). Also SF requires the force density for each element and the type of constraint that is applied to the particular element. ATLAS treats the SF as a separate module (subroutine) and the exchange of any data is passed through the argument list.

The second step is the assembly of the force density matrix (D). The force density matrix (Equation 2.11) contains the force densities of the cables that are connected to the cable movable nodes and it is discussed in more detail in Chapter 2. When the force density matrix is assembled, SF assembles the other part of Equation 2.12 i.e. $[(p_x - D_f x_f),(p_y - D_f y_f),(p_z - D_f z_f)]$. Once all the parts are assembled, SF solves the Equation 2.12 for the new nodal coordinates.

The next step is used to calculate the tensions in each cable and assemble G^t which is discussed in Chapter 2 and shown in Equation 2.16. When G^t is assembled the SF calculates the vector with the unbalanced external constraints, r, as shown in Equation 2.17 and checks for convergence i.e. SF checks whether the entries in the unbalanced vector, r, fall within the acceptable tolerance limits. If the process has converged to a solution the SF exits. If the process has not converged to the solution, SF calculates the increment values of the force densities, Δq, by solving Equation 2.17. The next step of the FDM is to update the force densities to be used in the next iteration. This is shown in Equation 2.13. However, before the update of the force densities the MALR is implemented. MALR is a convergence method and it is discussed in an earlier section of

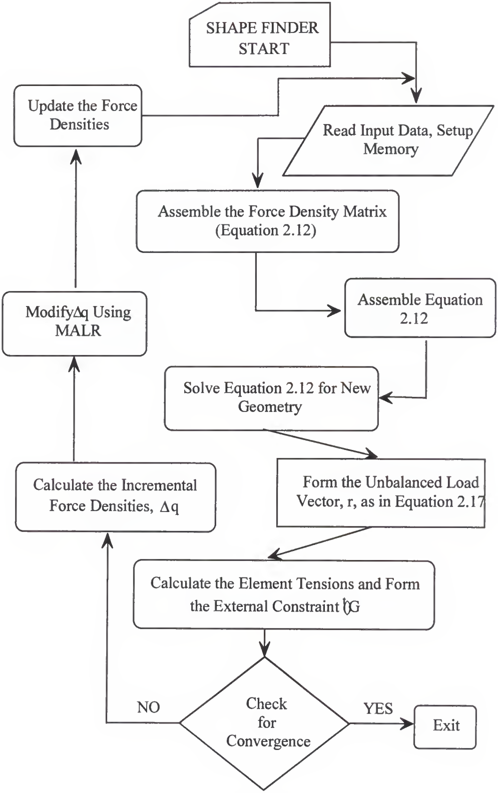

Figure 5. 3 Flowchart of the Shape Finder (Force Density Method)

this chapter. In this stage the increment values of the force densities are modified based on their history pattern. Then the modified increment values of the force densities are used to update the force densities to be used in the next iteration of the force density. When the SF exits, in ATLAS software, it returns the new geometry of the cable network and the new cable tensions in to keep the cable network in equilibrium.

5.6.2 DSM solution and NLDSM solution

The implementation of the DSM-NLDSM solution is shown in Figure 5.4. The program is separated in two parts as shown on the figure. The first part is referred to as the DSM solution and it performs a linear analysis for the space frame. The second part is referred to as the NLDSM solution and it performs a nonlinear analysis of the space frame including large displacements. The first step is to obtain the input data and allocate the memory to the required data structures. The input data consists of the information about the nodal points and the information about the frame elements. The nodal point information consists of the geometry of the nodal points as well as their description. Unlike the Shape Finder which refers to fixed nodes or movable nodes, the boundary conditions for the nodal points describe the ability of a node to displace in a particular direction in space. Therefore, for each node the program requires six parameters for the boundary conditions. The first three refer to the ability of the node to translate in the direction of the x, y, z axes respectively and the next three refer to the ability of the node to rotate about x, y, z respectively. Similarly to the previous program *'F'* is abbreviated for the restrained (fixed) displacement and *'R'* for the released. The element input data consists of the element connectivity i.e. the end nodes of each element (each element is connected to two nodes only) and the element properties. The frame element properties consist of the Young's modulus of elasticity, E, the cross sectional area, A, the moment of inertia, I, the torsional moment of inertia, J, and the element shear modulus, G. ATLAS

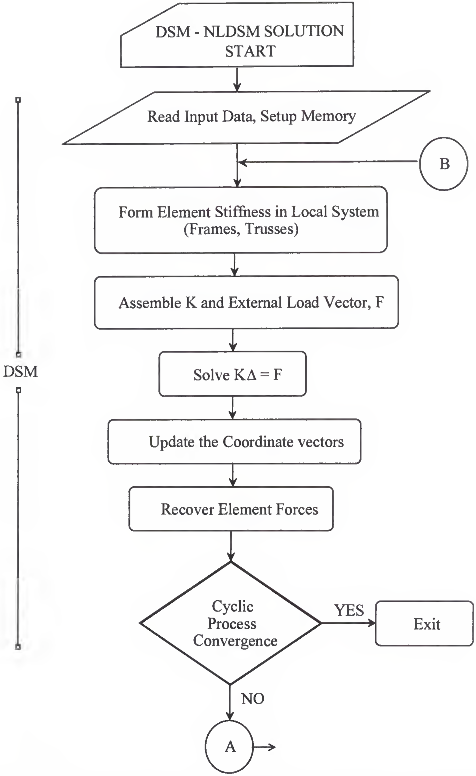

Figure 5. 4 Flowchart of the Direct Stiffness Method (Continued on next page)

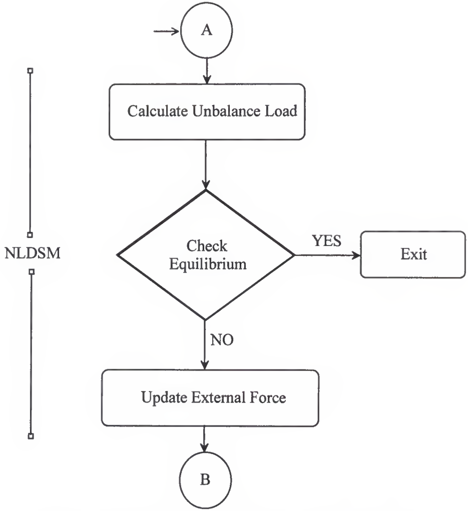

Figure 5. 4 (cont) Flowchart of the Direct Stiffness Method

treats the DSM and NLDSM solution as a separate module (subroutine) and the exchange of any data is passed through the argument list.

The second step is the assembly of the element stiffness in the local coordinate axes. The element stiffness matrix is shown on Figure 4.3. Once the element stiffness matrix is obtained the program assembles the global (structure) stiffness matrix which is a superposition of the individual element stiffness by means of transformation matrices and is shown in Equation 4.1. Next the applied load vector which is shown in Equation 4.2, is assembled. The load vector for the DSM solution consists of the tensions in the cable network. The tensions are obtained from the Shape Finder. The load vector for the NLDSM solution consists of the external loads that are applied on the structure. The solution of Equation 4.2 is the next step to be performed. The results consist of the nodal displacements. When the displacements are obtained the space frame geometry is updated as well as the element forces. This is the end of the DSM solution. If the cyclic process for the calculation of the space frame displacements has not converged to an approximate solution the program exits. The cyclic procedure is discussed in detail in an earlier section of this chapter. If on the other hand the cyclic process has converged to an approximate solution then the NLDSM part of the solution is implemented.

The NLDSM solution is implemented for the analysis of the structure as a whole. Therefore, the two substructures are superimposed to form the complete structure. The load on the structure consists of the true applied load i.e. the cable tensions are not applied as external loads on the frame elements. The NLDSM solution performs a few extra steps. First it calculates the unbalance load and then it checks for convergence of the forces. If the unbalance load is within acceptable tolerance the program exits. If on the other hand the unbalance load is greater than the acceptable tolerance then the program sets the external load vector equal to the unbalance load and performs the next iteration. Note that no convergence methods are applied in the NLDSM solution even if this is an

iterative technique. The reason is that the approximate solution that is obtained from the cyclic process is very close to the true solution. Therefore, the number of iterations in the NLDSM solution is minimum and no convergence method is necessary to be implemented to accelerate the process.

5.6.3 ATLAS Software

The ATLAS software is developed to implement the technique that is presented in this work. ATLAS is used to analyze the signal light and sign supports which consist of space frames and cable networks as shown in the examples in Chapter 6. The flowchart in Figure 5.5 shows the technique as implemented in ATLAS.

The flowchart is organized in stages which are executed successively. Each stage consists of a group of commands i.e. each major step of this technique represents a separate stage. The four major steps of this technique are clearly shown on the flowchart. For the sake of clarity the particular details of each step are purposely omitted from the figure since they are discussed in earlier sections of this chapter. The flowchart also shows the transition stages which are required for the transfer of data between successive stages.

The first stage refers to "major step 1" of the analysis as discussed earlier in the chapter. In this step ATLAS reads the problem data from an input data file. The data consists of the structure geometry, the applied load and the element information. The input data is discussed in more detail in the previous sections. The input data is organized in different data structures which are used to setup the problem, differentiate the substructures and specify the connectivity of the Frame-Cable (FC) structure. The organization of the input data in a certain way is necessary because in subsequent stages the analysis of the structure is based on the separate analyses of the two substructures as

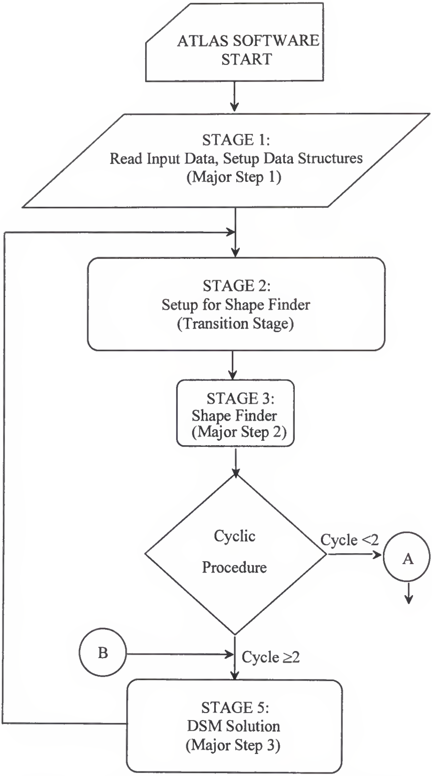

Figure 5. 5 Flowchart of ATLAS (continued on next page)

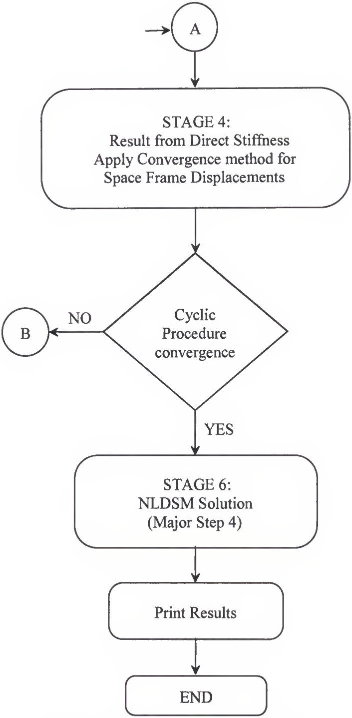

Figure 5. 5 (cont) Flowchart of ATLAS

mentioned in earlier sections. Therefore, the data is organized in such a way so it can be transferred to the different subroutines which perform the analysis of the substructures.

The second stage is a transition stage. During this stage ATLAS prepares the data to send to the Shape Finder. The information of the cable network which includes the nodal point information, the load information and the cable element information is distinguished from the rest of the structure. The third stage refers to "major step 2" of the analysis and it is used to obtain the shape of the cable network. The Shape Finder is implemented to carry out this task.

Stage four is a transition stage. It is used to setup the problem for the next stage which is the DSM solution. The magnitudes of the tensions to be applied on the space frame are specified in this stage as well as their directions. Their directions depend on the geometry of the cable network which is obtained in stage three. The magnitudes of the applied tensions are modified based on the convergence method for the space frame displacements which was discussed earlier in the chapter. Finally in this transition stage there is a check for the convergence of the space frame displacements. Whether the cyclic process has converged to a solution or not, is decided in this stage.

The next stage as shown on the flowchart is based on the outcome from stage four which is whether there is a need for the cyclic process to continue or not. If the process is not converged for the space frame displacements, the cyclic process will continue with stage 5. This stage consists of the DSM solution which refers to "major step 3". The space frame problem is already setup from stage four, so the DSM can execute directly to obtain the space frame displacements and stresses. When these are obtained, the process cycles back to step two to setup the problem for the force density solution of the next cycle. The space frame displacements are accounted to specify the locations of the cable base nodes.

If on the other hand the cyclic process converges to a solution for the space frame displacements the NLDSM solution is executed. This is shown as stage six on the flowchart and it refers to “major step 4” of the technique. In stage six the whole FC structure is considered and the out of balance forces are determined. As previously mentioned the magnitudes of the out of balance forces are expected to be minimal.

Stages seven and eight are performed after the analysis is complete. Stage seven is a design stage which is not discussed and stage eight is the printout of the results.

The technique that we have just described is dependable. This is expected because it involves two different but stable techniques which even if they are executed successively they are executed separately without actually affecting the execution of each other. The technique is also very efficient. This is owed to the fact that the FDM is very fast. Recall that during the FDM there is no need for stiffness matrix assembly. There are also cases where the technique may become inefficient. This happens with the presence of large out of balance forces and consequently with the necessity for many iterations in step six or “major step 4”. Such case is simple to avoid by applying tight tolerance values for the cyclic procedure (FDM - DSM solution) convergence.

CHAPTER 6
THE DUAL CABLE SYSTEM - ATLAS VERIFICATION - EXAMPLES

6.1 Introduction

The category of Frame-Cable structures is very broad. The flexible nature of these structures complicates their analysis which becomes very tedious and computationally expensive. In addition to that, each of these structures has its own characteristics based on architectural limitations, the construction sequence, the applied loads etc. The solution strategy which is presented in the previous chapters of this dissertation serves as a tool for the analysis of general Frame-Cable structures. In order to analyze a particular Frame-Cable structure however, the solution strategy must be adapted to the peculiarities of the particular structure. This chapter presents the necessary adjustments for the analysis of the Signal Lights and Signs supported by the Dual-Cable System. Also provided, are some examples of such structures which are analyzed using the ATLAS software. ATLAS was developed for this particular reason.

6.2 The Dual Cable-Supported System

6.2.1 The Structure

A typical dual cable-supported system is shown in Figure 6.1. The structure is typically planar and consists of prestressed concrete (or steel) poles, cables, connector (hanger) elements and signal lights and/or signs. The term planar is used loosely to denote the inability of the structure to provide out of plane stiffness. The top cable (catenary)

which is referred to as the "Primary Cable" supports the loads that are imposed on the structure by the self weights of the individual signal light and sign elements. The bottom cable (messenger) which is referred to as the "Secondary Cable" is used to constrain the lateral movement of the lights which is caused by the wind loads that are applied on the structure. Finally, the poles support both cables and serve as the media for transferring loads to the foundation.

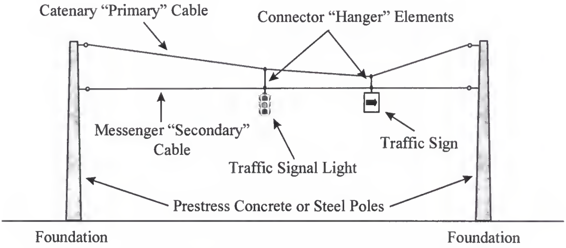

Figure 6. 1 Typical Dual Cable Supported System

The different types of elements that compose the structure under consideration generate a problem of unique nature. The uniqueness arises from the fact that each of the element types behaves in a different manner from the rest. When loaded, the flexible cable elements undergo large displacements which cause the geometry of the structure to change. Once the geometry changes, the geometric nonlinearities which are discussed in Chapter 3, are induced. Therefore, the cables are treated as elements with geometric nonlinearities. The tensions that are induced in the cables are transferred to the rigid poles which are treated as linear columns (frames). The last type of elements that makes up the structure is the signal lights and/or signs and the hanger/connector elements. These elements are supported by the cables. Therefore, when the cables displace, the connectors

and the signal lights displace as well. This motion, however, is mostly rigid body because there are no large strains involved. Therefore, there is no material nonlinearity in the connectors. These elements are treated as linear frames subjected to large rigid body motions.

6.2.2 Construction process

The first step in the construction sequence is the erection of the poles. Then the bottom cable (messenger) is attached to the poles. The height of the messenger cable is determined by the roadway clearance. When the bottom cable is in place, the lights are attached to the top (catenary) cable which is then lifted to obtain the required spacing between the cables (about 6% of the cable span); usually the smaller distance between the two cables is about 1 ft. When the catenary cable is installed, the signs and/or signal lights are attached to the messenger cable. Since both the top and bottom cables are stretched to their initial position they develop an initial tension. This kind of arrangement causes the catenary cable to carry all of the signal and/or sign weights. The role of the messenger cable is not significant when the wind forces are not present. However, when the wind forces are applied, the bottom cable picks up substantial load as it acts to prevent excessive swing.

6.2.3 Loads

There are three different kinds of loads that are applied on this kind of structures.

1. The prestress force in the cables.
2. The gravity loads which consist of the element weights.
3. The wind loads.

The prestress forces in the cables are induced during the construction period and they balance the applied loads when the structure is erected. In the case of the dual cable

supported system these loads consist of the gravity loads of the cables, the hangers and the signal lights and/or signs. The element weights (connectors, signal lights and/or signs) act as "free" weights hanging from the catenary cable. The gravity loads are basically supported by the catenary cables since the messenger cables are horizontal and therefore they do not have a downward force component. The wind load is applied on the centroid of the signs and/or the signal light heads and it is a function of the wind speed and the projected element surface areas.

6.2.4 Analysis of the Dual-Cable System

The Dual-Cable System can be categorized as a Frame-Cable structure since it consists of cables and frames. Therefore, when the cable and the frame substructures are defined, the system can be analyzed using the solution strategy that is presented in this dissertation. The analysis of this type of systems is obtained with the aid of the ATLAS software and it involves the analysis of the system for two different load cases:

1. The gravity solution when only the gravity load is applied.
2. The complete solution when all the load is applied (gravity, wind).

The gravity solution deals with the analysis of the structure when it is first erected and there is no other load but its own weight applied on it. The analysis involves the determination of the required shape of the structure so that it meets the necessary functionality requirements as well as the forces and stresses that are developed in the elements due to the gravity loads. The functionality of the system depends upon the distance from the road surface to the lowest point of the signal lights and/or signs which is referred to as the clearance. The gravity solution specifies also the initial (stress free) state of the elements that compose the structure. This is especially important for the cable elements which are prestressed in the construction process and therefore their exact stress

free state is difficult to obtain. As it is already mentioned the gravity load for this system is carried by the primary cables.

In order to obtain the required shape, ATLAS considers only the catenary cable and the CLEAR parameter which is specified by the user. When CLEAR is specified, ATLAS performs a series of Force Density analyses until that parameter is obtained. The CLEAR parameter as well as the structure that is used to obtain the necessary shape are shown on Figure 6.2. The first analysis which is used as reference, is obtained based on the input data which consists of the initial sag of the catenary cable and its weight per unit length. The second analysis is obtained for 50% of the initial cable sag. The cable weight per unit length is obviously the same. When the second analysis is obtained the program calculates the lowest point on the cable and compares it to the CLEAR parameter. If the CLEAR parameter is not obtained then the cable sag is modified and the next analysis is executed. The cable sag is modified based on linear interpolation between the cable sag of the previous two solutions. When the CLEAR parameter is obtained the Direct Stiffness Method is implemented to complete the gravity solution. At the end of the gravity solution the initial state of each element can be calculated from the final shape of the structure and the stresses in the element. Also at the end of gravity solution, ATLAS calculates the initial sag that is required for the catenary cable in order to obtain the required shape after the application of all the gravity load.

The wind load solution deals with analysis of the system when all the load is applied on it. In order to obtain the wind solution the structure as a whole has to be considered. The presence of the connector and the signs and/or signal light elements however, complicates the definition of the cable and frame substructures. In order to obtain the approximate solution (see Chapter 5) only the poles, the cables and the connector (hanger) elements are considered. Therefore, the presence of the signs and/or signal light elements is omitted in the calculation of the approximate solution. It turns out

that this assumption is not critical to the final result if the necessary adjustments are made. These adjustments are discussed in a later section. The pole elements are considered as the frame substructure (Figure 6.3a). The primary and secondary cables together with the connector elements are considered to be the cable network substructure as shown in Figure 6.3b. Therefore, during the Force Density solution, the connector elements are treated as cables which support axial forces only.

When the gravity solution is obtained then the next step in the analysis of these type systems is to consider any additional load. In the case of the dual cable supported system, this is the load due to the wind. The wind load on this structure is assumed to be applied at the centroid of the signs and/or signal lights and as previously mentioned it is a function of the wind speed and the element projected surface areas. The wind force on a sign or a signal light is applied in two forms, the drag force and the uplift and it is calculated using Equation 6.1 below:

$$F = C.P.A \tag{6.1}$$

where:

F = Wind Force on the element Centroid in pounds *(lb)*.
C = Wind Load Coefficient.
P = Wind Pressure in pounds per square foot *(psf)*.
A = Element Projected Area in square feet $\left(ft^2\right)$.

The wind pressure over the element area is a function of the wind velocity and it is calculated based on Equation 6.2 which is shown below:

$$P = 0.00256\left(1.3V\right)^2 \tag{6.2}$$

where:

V= Wind Velocity in miles per hour *(mph)*.

The wind force on the element is applied in two forms, the drag force and the uplift. The wind load coefficient is used to define the two different forces. The drag force

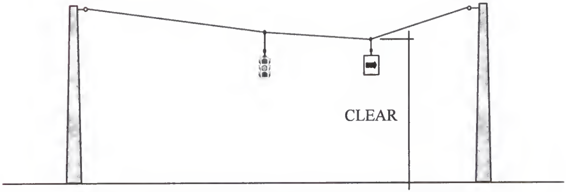

Figure 6. 2 Part of the System Used to Obtain the Required Shape

(a)

(b)

Figure 6. 3 Frame-Cable Substructures

coefficient (C_D) is used to obtain the drag force whereas the uplift coefficient (C_L) is used to obtain the uplift. The value of the two coefficients is obtained from Equations 6.3 and 6.4 [19] which represent the best fit polynomials for the charts shown below in Figure 6.4 and Figure 6.5.

$$C_D = 1.2 - 1.08*10^{-2}\,\theta - 4.16*10^{-5}\,\theta^2 \tag{6.3}$$

$$C_L = 2.32*10^{-2}\theta - 1.87*10^{-4}\,\theta^2 - 2.23*10^{-6}\,\theta^3 + 8.61*10^{-8}\,\theta^4 - 7.79*10^{-10}\,\theta^5 \tag{6.4}$$

where:

θ = Rotation Angle in Degrees (^{o}).

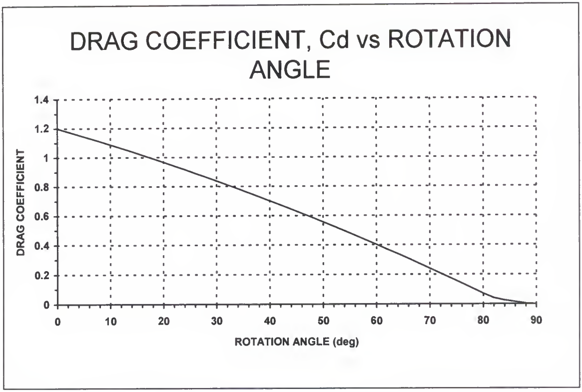

Figure 6. 4 Plot of Drag Coefficient

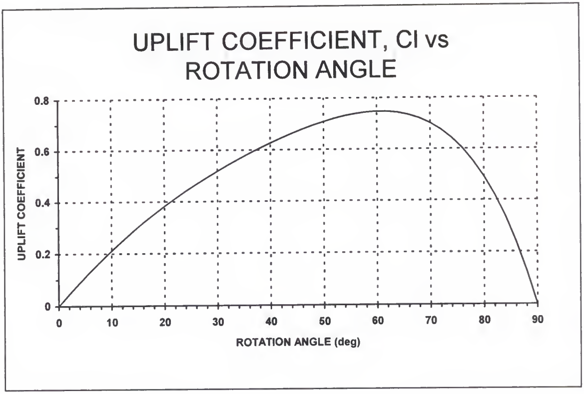

Figure 6. 5 Plot of Uplift Coefficient

The wind force that is applied on the structure corresponds to the maximum wind speed that the structure is expected to experience. It is assumed to be static and does not change with time. However, the wind force changes "dynamically" since it is a function of the rotation angle of the element. Therefore, when the load is applied on a particular element the element displaces, causing the wind load to change. This complicates the implementation of the FDM solution since Force Density Method assumes that the applied load remains constant throughout the analysis. In order to handle the load changes ATLAS performs a series of Force Density analyses before it completes one FDM solution. The initial structure remains the same in each of these analyses but the applied load changes. The applied load for each of these analyses corresponds to a different rotation angle for the element. The final solution is obtained when the initial rotation angle that the applied load is based on, is equal to the rotation angle of the element after the analysis is completed. So the FDM solution as discussed in Chapter 5 needs to be

modified to allow for the series of Force Density analyses. The term "windcycle" is used in the remainder of this chapter to describe the analysis that is obtained from the Force Density analysis for a particular load. Therefore, one FDM solution consists of several windcycles.

Figure 6.6 shows the flowchart of the modified FDM solution as implemented in ATLAS. At the top of each windcycle the load is obtained based on a trial rotation angle and the Force Density Method is implemented to carry out the analysis. When the analysis is completed, ATLAS calculates the element's rotation angle and compares it with the one at the top of the windcycle. If the two angles are the same (within acceptable tolerance), the FDM solution is considered to be complete and the program proceeds for the DSM solution. If on the other hand the two angles do not match then there is a need for more windcycles and ATLAS cycles back. The rotation angle is updated at the top of the windcycle and the new solution is obtained. In simpler words ATLAS projects the resulting rotation angle and calculates the wind load based on that angle. If the projected rotation angle is not good then the program projects another rotation angle to be the solution and performs another windcycle. The process is repeated until the final solution is obtained.

The projected values for the rotation angles between successive windcycles are very important since the stability as well as the efficiency of the process depend on it. Several attempts were made to minimize the execution time. It turned out that the simplest technique is the most stable. Studying the values of the wind load coefficients which are shown on the charts on Figure 6.4 and Figure 6.5, one realizes that the applied wind load becomes zero when the rotation angle of the element becomes 90° which implies that the solution of the Force Density analysis is always in the range of zero to ninety degrees (0° - 90°). Therefore, ATLAS starts by assuming that the rotation angle is

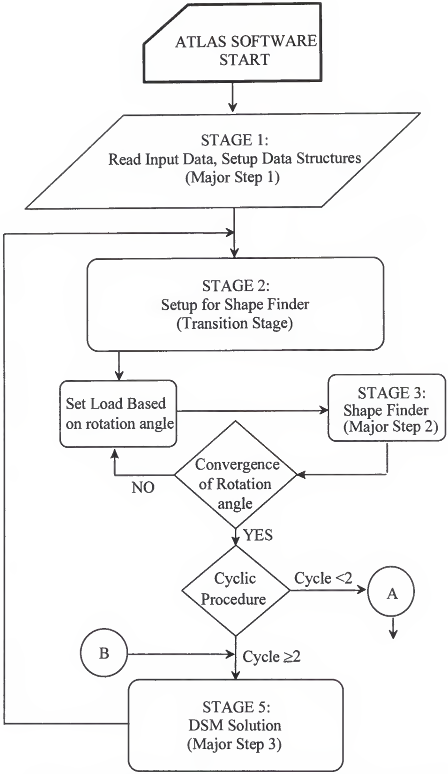

Figure 6. 6 Modified ATLAS Flowchart (Continued on next page)

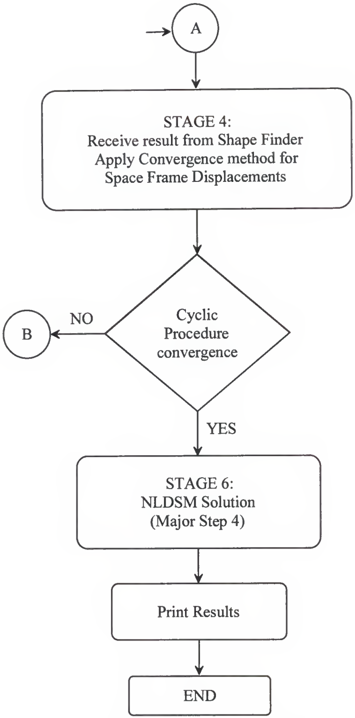

Figure 6. 6 (cont) Modified ATLAS Flowchart

90° and calculates the applied load based on that angle. If the analysis results to any rotation angle for the element that is different than 90° then the value of the rotation value is lowered by a specified decrement and the next windcycle is performed. The process is repeated until the rotation angle converges to a solution. The solution represents the end of a FDM solution as described in Chapter 5.

When the FDM solution is obtained then the DSM solution is performed to update the frame substructure displacements (in this case the pole displacements). The solution strategy continues as discussed in Chapter 5 until the final solution is obtained.

6.3 ATLAS Verification - Static Load Test

It was considered very constructive to use static loads to verify the computer program before any attempt was made to use ATLAS for the analysis of variable loads. This section presents a sample static load test that was performed at the University of Florida Civil Engineering Structures Laboratory. The static loads were calculated based on a certain wind speed. The applied loads were directly applied on the cable nodes and remained constant throughout the analysis. Therefore, the loads that were used were neither functions of the rotation angles of the elements nor functions of time.

6.3.1 Test System Configuration

The test system configuration for the static load test was to be representative of a full scale two-point connection dual cable system. The span of the system was selected to be 50 *ft*, comparable to a perpendicular system spanning a two-lane road with a turn lane and sidewalks on each side. Only one signal was modeled during the static load test in order to simplify the instrumentation as well as the loading requirements. The height of

the test system was set lower than that of an actual system for convenience in working with the system.

6.3.2 Strain Poles

The type of strain pole most commonly being used throughout Florida to support traffic signal and sign cable systems is the cantilever concrete strain pole. The use of this strain pole type was not practical for the static load test because it was performed in an indoor testing lab. For this reason, steel strain poles which could be easily erected in the lab were designed and fabricated. The strain poles were fabricated from wide flange section as opposed to a material such as square structural tubing, which would more closely model a concrete strain pole, because it was easier to obtain locally and less costly. The poles were braced with angle sections in both the strong and weak axes at a point slightly below the messenger cable connection. The purpose of the bracing was to minimize effects on the cable system due to strain pole deformations and therefore isolate the factors which influence the cable system behavior to the applied loads and the cable properties. It was anticipated that the fewer the factors affecting the cable system behavior, the less complicated it would be to. determine the cause of any inconsistencies between the laboratory test and ATLAS analysis results.

6.3.3 Cable, Cable Connection Devices, Signal Hardware

The components of the traffic signal system were provided by the Traffic Engineering Division. The components for the static load tests included the following items:

1. 1/4 *in* diameter zinc coated steel wire strand for use as the messenger cable.
2. 3/8 *in* diameter zinc coated steel wire strand for use as the catenary cable bales (also known as dead-ends) for attachment of the cables to the strain poles.

3. 1-1/2 *in* diameter pipe hanger saddle for attachment of the pipe hanger to the catenary cable bracket for attachment of the pipe hanger to the messenger cable.

6.3.4 Modeling of Static Load Test System for ATLAS Analyses

The information required in an ATLAS input data file consists of the system geometry and boundary conditions, material and section properties of the system components, and the loads applied to the system.

Nodal coordinates and boundary conditions. The first step taken in the preparation of an ATLAS input file was to assign coordinates and boundary conditions to the test system's nodes. The assignment consisted of converting the measured test system dimensions to a Cartesian coordinate system. Figure 6.7 illustrates the dimensions and node numbering of the test system.

The boundary conditions of each node were defined by specifying its degrees of freedom (DOF) as fixed or released. Nodes located at the strain pole and bracing bases - Nodes 1, 9, 12, 13, 14, and 15 - were specified as having fixed x, y, and z translational DOF and released Θx, Θy, and Θz rotational DOF. The rotations at these connections were not true pin connections and therefore not fully released rotationally, however it is believed that they behaved more like pin connections than fixed connections. All DOF for the remaining nodes were released since their movements were not restrained in any way.

Clear parameter. The *CLEAR* parameter defines the distance from the '$z = 0$' plane to the lowest position of the catenary cable after all dead loads have been applied. For all of the static load test systems the lowest position of the catenary cable was the point where the hanger was attached since this was the only point at which dead load in addition to the cable self-weight was applied.

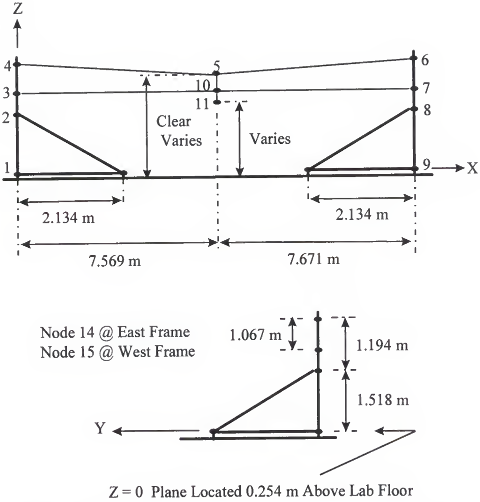

Figure 6. 7 Static Load Test System Dimensions and Node Numbers

Material and section properties. Material and section properties for the strain poles, bracing, cables, and hanger were required in the ATLAS input file. The required properties consisted of cross-sectional area (A) and modulus of elasticity (E) for all components and moment of inertia (I_2 *and* I_3) and torsional constant (J) for all of the components except the cables.

The material and section properties for the strain poles and bracing were obtained from the *AISC Manual of Steel Design: Load and Resistance Factor Design (LRFD)* [19]. The LRFD manual states that the modulus of elasticity for steel at 70 *°F* is approximately 29,000 *ksi,* therefore this value was used for the ATLAS analyses. Section properties for

the strain pole and bracing elements were obtained from the appropriate tables in the *LRFD* manual. The material and section properties for the hanger, constructed of 1-1/2 *in* nominal diameter standard weight pipe, were also obtained from the *LRFD* manual.

The catenary and messenger cable material was zinc coated seven wire strand, manufactured in accordance with *ASTM A475: Standard Specifications for Zinc-Coated Steel Wire Strand* [20]. The only required cable section property, cross-sectional area, was obtained from *ASTM A475* in terms of the nominal diameter of the coated wires in the strand. The 1/4 *in* messenger cable has a specified nominal coated wire diameter of 0.080 *in*, resulting in a cross-sectional area of 0.035 in^2 when the coated wire diameter is multiplied by seven, the total number of coated wires. The cross-sectional area of the 3/8 *in* catenary cable was computed in the same manner, with the coated wires having a specified nominal diameter of 0.120 *in*, resulting in a cross-sectional area of 0.079 in^2. The modulus of elasticity of the cables was estimated to be 23,000 *ksi*. This value was later confirmed to be within an acceptable range based on test data from Florida Wire and Cable, Inc.

Loads. The required format of an ATLAS input file dictates that the loading pattern applied to a system be divided into three data blocks - *SIGNS*, *LOADS*, and *WIND*. A general description of how the static test loading patterns were modeled for the ATLAS analyses follows.

The *SIGNS* block of the ATLAS input file defines the dead weight loads applied to the catenary cable before the signal hanger is attached to the messenger cable. These are the loads that are used to determine the geometry of the catenary cable before any other loads, such as wind loads, are applied to the system. The *SIGNS* load for the static load tests was applied to Node 5 and consisted of the hanger weight, the catenary cable weight within the tributary length of Node 5, and any concentrated load applied to the hanger before it was connected to the messenger cable. The messenger cable weight is not

included as a dead load because it is accounted for with the initial messenger cable tension parameter.

The *LOADS* block of the ATLAS input file defines the dead weight loads applied to the system after the initial catenary cable geometry has been determined by ATLAS. It includes the same dead weight loads as the *SIGNS* block, but these loads are applied to the actual nodes on which they act instead of being applied only to the catenary cable. For the static load test model, the concentrated load was applied to Node 12 and the saddle and catenary cable weights were applied to Node 5. The uniform hanger weight was distributed to the nodes common to the hanger - Nodes 5, 10, and 11 - based on the tributary hanger length of each node.

The *WIND* block of the ATLAS input file defines any loads applied to the system after the dead weight loads have been applied. These loads remain constant throughout the analysis. For the static load test model, this block contained the lateral loads and any vertical loads applied to the system at Node 11 after the hanger was attached to the messenger cable. The *WIND* block also included small lateral loads applied to the catenary and messenger cables at Nodes 5 and 10 due to the 0.866 *lb* pull of the potentiometer attachment cables.

Table 6.1 provides a breakdown of the loads applied to the test system. The magnitudes of the concentrated vertical and lateral loads varied between the tests and are noted as such in the table. Dead loads due to the test system components were determined by weighing them in the lab with the exception of the catenary cable weight, which was obtained from *ASTM A475*.

6.3.5 Static load test results

This section presents the static load test results [20]. A description as well as tabular and graphical comparisons of the test and ATLAS analytical results follow. The

next sections provide an evaluation of the comparison between the test and ATLAS analysis results and a discussion of possible reasons for any inconsistencies between them.

Static Load Test. The particular Static Load Test was performed on the test system as it was originally installed by the Traffic Engineering Division. The initial laboratory cable conditions after application of the system dead loads (the *SIGNS* loads) were a messenger cable tension of 1,129 *lb*, a catenary cable tension of 405 *lb*, and a *CLEAR* parameter of 100.95 *in*. The loading pattern applied to the test system consisted of the following steps:

1. The hanger was attached to the catenary cable and a concentrated vertical load of 35.4 *lb* was applied to it at a point 46-3/8 *in* below the catenary cable.
2. The hanger was attached to the messenger cable.
3. A concentrated lateral load of 29.0 *lb* was applied to the hanger at the same point the concentrated vertical load was applied.
4. The lateral load was increased to 56.3 *lb*.
5. The lateral load was increased to 83.6 *lb*.
6. The lateral load was increased to 101.2 *lb*.

Tables 6.1 through 6.8 and Figures 6.8 through 6.11 present the test and ATLAS analysis results.

6.3.6 Evaluation of Static Load Test Results

This section provides an evaluation of the static load test results and their comparison to ATLAS analysis results. The parameters for which test and ATLAS analysis data were presented - cable tensions, lateral cable displacements, vertical cable displacements, and connector rotations - are individually evaluated with regard to the extent of inconsistencies between the test and ATLAS analysis results and the suspected

Table 6.1 Static Load Test Applied Loads

Load Origin	Load Magnitude
SIGNS Block @ Node 5: Catenary Cable Weight Hanger Saddle Weight Hanger Weight Concentrated Vertical Load	 (0.27 lb/lf)(25 ft) = 6.75 lbs 2.65 lbs 13.9 lbs Varies - Provided in Next Section
LOADS Block @ Node 5: Catenary Cable Weight Hanger Saddle Weight Hanger Weight @ Node 10: Hanger Weight @ Node 11: Hanger Weight Concentrated Vertical Load	 (0.27 lb/lf)(25 ft) = 6.75 lbs 2.65 lbs (0.217 lb/in)(Variable Tributary Length) (0.217 lb/in)(Variable Tributary Length) (0.217 lb/in)(Variable Tributary Length) Varies - Provided in Next Section
WINDS Block @ Nodes 5 and 10: Potentiometer Attachment Cable Pull @ Nodes 11: Concentrated Lateral Load	 0.866 lbs Varies - Provided in Next Section

Table 6.2 Catenary Cable Tensions - Static Load Test

Lateral Load (lbs)	Test Value (lbs)	ATLAS Value (lbs)	Absolute Difference (lbs)	Percent Difference
0.0	405	405	0	0.00%
29.0	454	446	8	1.8%
56.3	528	530	2	0.38%
83.6	632	622	10	1.6%
101.2	696	680	16	2.4%

Table 6.3 Messenger Cable Tensions - Static Load Test

Lateral Load (lbs)	Test Value (lbs)	ATLAS Value (lbs)	Absolute Difference (lbs)	Percent Difference
0.0	1129	1129	0	0.00%
29.0	1227	1213	14	1.2%
56.3	1388	1374	14	1.0%
83.6	1610	1549	61	4.0%
101.2	1718	1659	59	3.6%

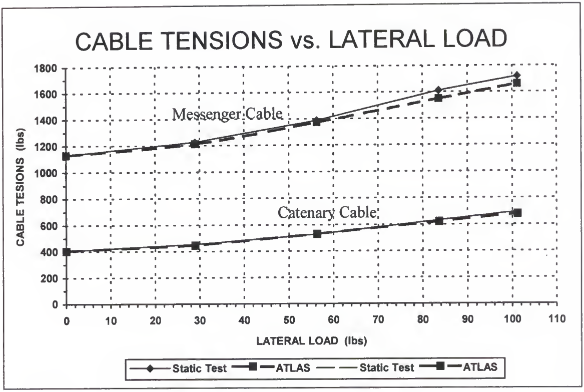

Figure 6. 8 Cable Tensions vs Lateral Force

Table 6.4 Catenary Cable Lateral Displacements - Static Load

Lateral Load (lbs)	Test Value (in)	ATLAS Value (in)	Absolute Difference (in)	Percent Difference
0.0	0.000	0.000	0.000	0.00%
29.0	1.668	1.692	0.024	1.5%
56.3	2.532	2.485	0.047	1.9%
83.6	2.916	2.727	0.189	6.5%
101.2	2.700	2.719	0.019	0.69%

Table 6.5 Messenger Cable Lateral Displacements - Static Load Test

Lateral Load (lbs)	Test Value (in)	ATLAS Value (in)	Absolute Difference (in)	Percent Difference
0.0	0.000	0.000	0.000	0.00%
29.0	-4.140	-4.188	0.048	1.2%
56.3	-6.900	-7.059	0.159	2.3%
83.6	-9.300	-9.122	0.178	1.9%
101.2	-10.368	-10.185	0.183	1.8%

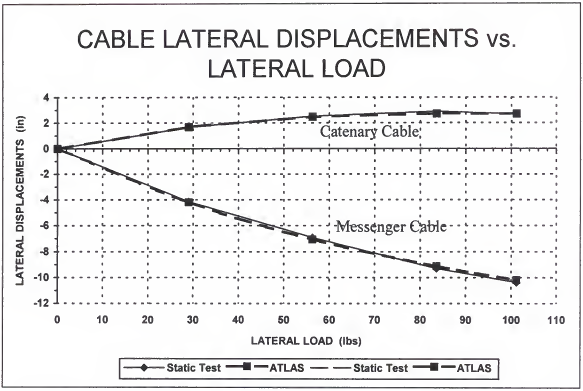

Figure 6. 9 Cable Lateral Displacements vs Lateral Force

Table 6.6 Catenary Cable Vertical Displacements - Static Load Test

Lateral Load (lbs)	Test Value (in)	ATLAS Value (in)	Absolute Difference (in)	Percent Difference
0.0	0.000	0.000	0.000	0.00%
29.0	0.188	-0.076	0.264	140%
56.3	-0.438	-0.263	0.174	40%
83.6	-0.813	-0.516	0.297	37%
101.2	-0.875	-0.688	0.187	21%

Table 6.7 Messenger Cable Vertical Displacements - Static Load Test

Lateral Load (lbs)	Test Value (in)	ATLAS Value (in)	Absolute Difference (in)	Percent Difference
0.0	0.000	0.000	0.000	0.00%
29.0	1.000	0.766	0.234	23%
56.3	2.219	2.079	0.140	6.3%
83.6	3.438	3.255	0.183	5.3%
101.2	3.781	3.889	0.107	2.9%

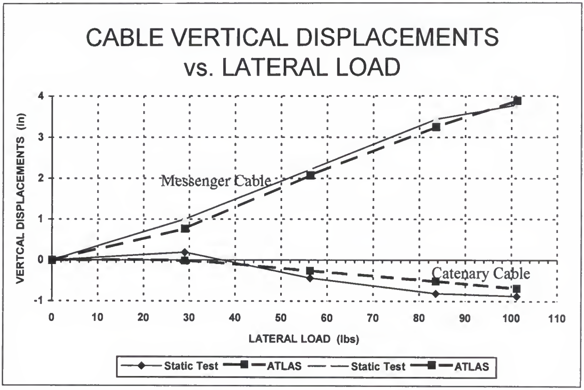

Figure 6. 10 Cable Vertical Displacements vs Lateral Force

Table 6.8 Connector Rotations- Static Load Test

Lateral Load (lbs)	Test Value (deg)	ATLAS Value (deg)	Absolute Difference (deg)	Percent Difference
0.00	0.00	0.00	0.00	0.00%
29.0	16.71	16.93	0.22	1.3%
56.3	27.84	28.20	0.36	1.3%
83.6	37.21	35.92	1.29	3.6%
101.2	40.31	39.70	0.61	1.5%

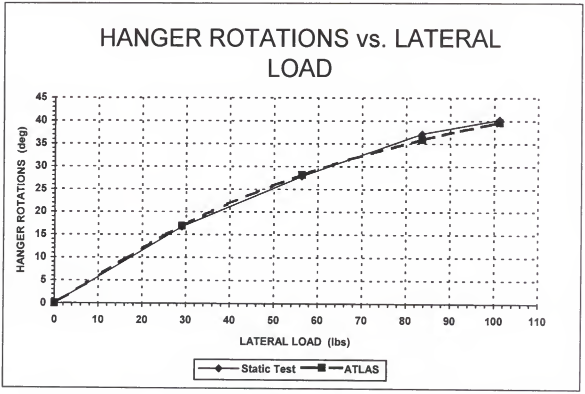

Figure 6. 11 Hanger Rotation vs Lateral Load

reasons for these inconsistencies. A thorough discussion of the possible reasons for inconsistencies between the results is provided in the following section.

Cable tensions. Comparisons between the static test and ATLAS cable tension values show that they match each other very well. The largest percent difference between the static test and ATLAS value is 2.4% for the catenary cable and 4.0% for the messenger cable. It is suspected that inconsistencies between these values are mainly due to differences between the cable modulus of elasticities which were used for the ATLAS analyses and the true cable modulus of elasticities. The test cable tension values are consistently larger than the ATLAS values, indicating that the true cable modulus of elasticities were greater than the 23,000 *ksi* value which was used for the ATLAS analyses. Inconsistencies may also be due to differences between the applied loads which were modeled for the ATLAS analyses and the true applied loads. These inconsistencies would likely be minimal but still noticeable.

Lateral cable displacements. Comparisons between the static test and ATLAS lateral cable displacement values show that the messenger cable values match each other well with the largest percent difference between them being 2.3%. The largest percent difference for the catenary cable is 6.5%. It is believed that part of the inconsistencies between these values resulted from a combination of reasons which includes variations between the applied loads and cable modulus of elasticities used for the ATLAS analyses and the true applied loads and cable modulus of elasticities, the method of distributing the uniform hanger weight for the ATLAS analyses, the laboratory initial hanger position not being truly vertical, and the method used to measure the lateral cable displacements. These reasons may fully explain the inconsistencies for the static load test, but the fact that lateral catenary cable displacement values are inconsistent may indicate that some additional reason exists for these tests. The most likely explanation is that some sort of slippage within the catenary cable connection occurred during these tests. This would

explain why the test catenary cable displacement values are consistently larger than the ATLAS values.

Vertical cable displacements. Comparisons between test and ATLAS vertical cable displacement values show that they do not consistently match each other as well as the values for the other parameters did. The catenary cable values are more inconsistent than the messenger cable values, partly because the vertical cable displacements are small and even a small absolute difference between the values results in a large percent difference. The largest absolute difference between any of the test and ATLAS values is only 0.297 *in* for the catenary cable and 0.234 *in* for the messenger cable. Some of the inconsistencies between the values may also be attributed to variations between the loads and cable modulus of elasticities used for the ATLAS analyses and the true applied loads and cable modulus of elasticities as well as the method used to measure the cable displacements.

Connector rotations. Comparisons between test and ATLAS connector rotations show that they match each other well. The percent differences between the test and ATLAS connector rotation values are all within 4%. Since the connector rotations are dependent on the lateral and vertical cable displacements, inconsistencies between the test and ATLAS values are a direct result of inconsistencies between the previously discussed cable displacement values.

Summary. The cable tensions and connector (or signal) rotations are the most important parameters concerning the operation of a traffic signal and sign systems when subjected to large lateral loads. It is therefore most important that the ATLAS analytical results for these parameters be verified by matching the test results since they are critical in the design procedure for traffic signal and sign support systems. Comparisons of the cable tension and connector rotation test and ATLAS analysis values show that they match each other very well for the static load test. The lateral and vertical cable

displacement values do not match each other quite as well as the cable tension and connector rotation values do. This is believed to have resulted from differences between the laboratory test system and the system modeled for the ATLAS analyses.

6.4 Examples

The following examples show the analyses of different dual cable supported systems as obtained by ATLAS. The required input data files which contain the necessary input data to describe the structures as well as the resulting output data files which contained the results of the analysis for each of the systems are provided. Each of the input data files contains nodal information which consists of the nodal geometry and boundary conditions. Also the input data file contains information about the different elements which consists of the element properties and connectivity. The applied loads that are applied on the structure are also provided.

The output data files provide an echo of the input data and the final results of the analysis. The final results consist of the final geometry of the structure and the forces in the elements that are developed to maintain equailibrium.

Example 6.1

The following example represents a typical two dimensional dual-cable supported system. The structure spans diagonally over an intersection as shown on Figure 6.12a. The required clearance is 17 *ft*. The catenary cable has 3/8 *in* diameter and the messenger cable 1/2 *in*. The poles are made of prestress concrete and they are both of type *NVII*. The signal lights and sign arrangement are also shown on Figure 6.12b.

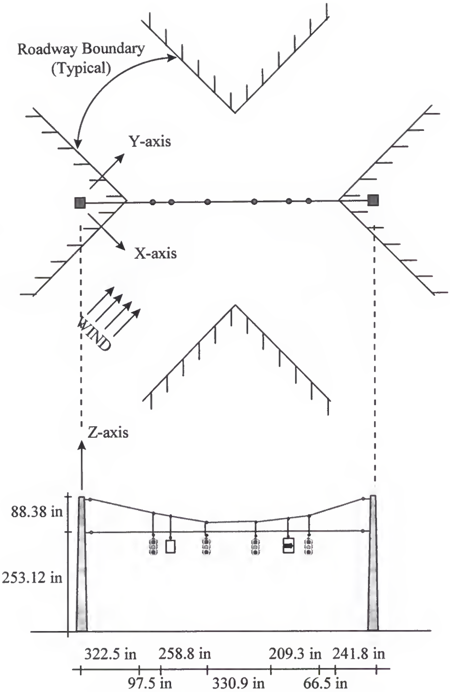

Figure 6. 12 Structure for Example 6.1

```
CONTROL
Title= Example 1 - Typical Intersection
NODES= 26
CLEAR= 265.12
CABLE=  2
SPEED=  60.0
ANGLE= 90
STATUS=3
:
CABLES
  1  3  7   S=    6.0   W=        0.23E-04   P=0
  2  2  6   T=    1.0   W=        0.43E-04   P=1
:
COORDINATE
     1 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.0000000E+00
     2 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.2531200E+03
     3 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.3414875E+03
     4 X= 0.6084854E+03 Y= 0.6084854E+03 Z= 0.0000000E+00
     5 X= 0.1680000E+04 Y= 0.1680000E+04 Z= 0.0000000E+00
     6 X= 0.1680000E+04 Y= 0.1680000E+04 Z= 0.2531200E+03
     7 X= 0.1680000E+04 Y= 0.1680000E+04 Z= 0.3414875E+03
     8 X= 0.1688485E+04 Y= 0.1688485E+04 Z= 0.0000000E+00
     9 X= 0.8277827E+03 Y= 0.8277827E+03 C=     1
    10 X= 0.8277827E+03 Y= 0.8277827E+03 Z= 0.2531200E+03
    11 X= 0.8277827E+03 Y= 0.8277827E+03 Z= 0.2250600E+03
    12 X= 0.8971569E+03 Y= 0.8971569E+03 C=     1
    13 X= 0.8971569E+03 Y= 0.8971569E+03 Z= 0.2531200E+03
    14 X= 0.8971569E+03 Y= 0.8971569E+03 Z= 0.2381200E+03
    15 X= 0.1078025E+04 Y= 0.1078025E+04 C=     1
    16 X= 0.1078025E+04 Y= 0.1078025E+04 Z= 0.2531200E+03
    17 X= 0.1078025E+04 Y= 0.1078025E+04 Z= 0.2250600E+03
    18 X= 0.1313402E+04 Y= 0.1313402E+04 C=     1
    19 X= 0.1313402E+04 Y= 0.1313402E+04 Z= 0.2531200E+03
    20 X= 0.1313402E+04 Y= 0.1313402E+04 Z= 0.2250600E+03
    21 X= 0.1462061E+04 Y= 0.1462061E+04 C=     1
    22 X= 0.1462061E+04 Y= 0.1462061E+04 Z= 0.2531200E+03
    23 X= 0.1462061E+04 Y= 0.1462061E+04 Z= 0.2381200E+03
    24 X= 0.1509136E+04 Y= 0.1509136E+04 C=     1
    25 X= 0.1509136E+04 Y= 0.1509136E+04 Z= 0.2531200E+03
    26 X= 0.1509136E+04 Y= 0.1509136E+04 Z= 0.2250600E+03
:
BOUNDARY
     1 DOF=f f f f f f
     2 DOF=r r r r r r
     3 DOF=r r r r r r
     4 DOF=f f f f f f
     5 DOF=f f f f f f
     6 DOF=r r r r r r
     7 DOF=r r r r r r
     8 DOF=f f f f f f
     9 DOF=r r r r r r
    10 DOF=r r r r r r
    11 DOF=r r r r r r
    12 DOF=r r r r r r
    13 DOF=r r r r r r
    14 DOF=r r r r r r
    15 DOF=r r r r r r
    16 DOF=r r r r r r
    17 DOF=r r r r r r
    18 DOF=r r r r r r
    19 DOF=r r r r r r
    20 DOF=r r r r r r
    21 DOF=r r r r r r
    22 DOF=r r r r r r
    23 DOF=r r r r r r
    24 DOF=r r r r r r
    25 DOF=r r r r r r
    26 DOF=r r r r r r
:
```

```
PRIMARY
    7,     1
  1 A=        0.0790 E=  0.27500E+05
   1      3,    9 M=  1 C=  1
   2      9,   12 M=  1 C=  1
   3     12,   15 M=  1 C=  1
   4     15,   18 M=  1 C=  1
   5     18,   21 M=  1 C=  1
   6     21,   24 M=  1 C=  1
   7     24,    7 M=  1 C=  1
:
SECONDARY
    7,     1
  1 A=        0.1500 E=  0.27500E+05
   1      2,   10 M=  1 C=  2
   2     10,   13 M=  1 C=  2
   3     13,   16 M=  1 C=  2
   4     16,   19 M=  1 C=  2
   5     19,   22 M=  1 C=  2
   6     22,   25 M=  1 C=  2
   7     25,    6 M=  1 C=  2
:
CONNECTORS
    6,     2
  1 A=        1.0700 E=  0.29000E+05 I=  0.66600E+00, 0.66600E+00  \
    J=  0.13300E+01 G=  0.00000E+00
  2 A=        0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00
   1      9,  10,    1 M=  1
   2     12,  13,    1 M=  2
   3     15,  16,    1 M=  1
   4     18,  19,    1 M=  1
   5     21,  22,    1 M=  2
   6     24,  25,    1 M=  1
:
LIGHTS
    6,     2
  1 A=        1.0700 E=  0.29000E+05 I=  0.66600E+00, 0.66600E+00  \
    J=  0.13300E+01 G=  0.00000E+00 S= 1 P=  0.59136E+03, 0.59136E+03
  2 A=        0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00 S= 0 P=  0.72000E+03, 0.00000E+00
   1     10,  11,    1 M=  1
   2     13,  14,    1 M=  2
   3     16,  17,    1 M=  1
   4     19,  20,    1 M=  1
   5     22,  23,    1 M=  2
   6     25,  26,    1 M=  1
:
BEAM
    4,     1
  1  T= NVII   FC=      6000.0
   1      1,   2,    4 M=  1
   2      2,   3,    4 M=  1
   3      5,   6,    8 M=  1
   4      6,   7,    8 M=  1
:
SIGNS
    9 F=  0.00000E+00, 0.00000E+00,-0.18109E+00
   12 F=  0.00000E+00, 0.00000E+00,-0.15643E-01
   15 F=  0.00000E+00, 0.00000E+00,-0.17843E+00
   18 F=  0.00000E+00, 0.00000E+00,-0.17919E+00
   21 F=  0.00000E+00, 0.00000E+00,-0.16360E-01
   24 F=  0.00000E+00, 0.00000E+00,-0.18170E+00
:
WIND
:
```

```
LOADS
   3 F=  0.00000E+00, 0.00000E+00,-0.36402E-02
   7 F=  0.00000E+00, 0.00000E+00,-0.27323E-02
   9 F=  0.00000E+00, 0.00000E+00,-0.89208E-02
  10 F=  0.00000E+00, 0.00000E+00,-0.89208E-02
  11 F=  0.00000E+00, 0.00000E+00,-0.16325E+00
  12 F=  0.00000E+00, 0.00000E+00,-0.58290E-02
  13 F=  0.00000E+00, 0.00000E+00,-0.58290E-02
  14 F=  0.00000E+00, 0.00000E+00,-0.39851E-02
  15 F=  0.00000E+00, 0.00000E+00,-0.66524E-02
  16 F=  0.00000E+00, 0.00000E+00,-0.66524E-02
  17 F=  0.00000E+00, 0.00000E+00,-0.16512E+00
  18 F=  0.00000E+00, 0.00000E+00,-0.72877E-02
  19 F=  0.00000E+00, 0.00000E+00,-0.72877E-02
  20 F=  0.00000E+00, 0.00000E+00,-0.16461E+00
  21 F=  0.00000E+00, 0.00000E+00,-0.66201E-02
  22 F=  0.00000E+00, 0.00000E+00,-0.66201E-02
  23 F=  0.00000E+00, 0.00000E+00,-0.31199E-02
  24 F=  0.00000E+00, 0.00000E+00,-0.98603E-02
  25 F=  0.00000E+00, 0.00000E+00,-0.98603E-02
  26 F=  0.00000E+00, 0.00000E+00,-0.16198E+00
:
```

The results of the analysis for the above structure are shown in the following data file which is created by ATLAS. The discussion that follows provides a detailed explanation of the different sections of the output data file. The data file that was chosen for this purpose is a typical output file that is obtained from the analysis.

The first part of the output file consists of the program title as well as a list of the input and output data files that are involved in the analysis of this particular structure.

```
****************************************************************************
|                                                                          |
|      ##         #####       #               ##          ####             |
|     #  #          #         #              #  #        #                 |
|    #    #         #         #             #    #        ####             |
|    ######         #         #             ######            #            |
|    #    #         #         #             #    #       #    #            |
|    #    #         #         ######        #    #        ####             |
|                                                                          |
|                Analysis of Traffic Lights And Signs                      |
|                                                                          |
|                             Version 4.0                                  |
|                                                                          |
|            Developed by :                                                |
|                                                                          |
|            Dr Marc I. Hoit               Dr Ronald A. Cook               |
|            Mr Petros M. Christou         Ms Adeola K. Adediran           |
|                                                                          |
|                  Department of Civil Engineering                         |
|                       University of Florida                              |
|                       Gainesville, Fl 32608                              |
|                                                                          |
****************************************************************************

 Input Data File = archer
 Output Data File = arc.out
```

In the following sections ATLAS echoes the data that is provided in the input data file (archer). This data consists of the data in the control block as well as the nodal point data (coordinates and boundary conditions). Also provided is the element material property information and connectivity. Finally the applied loads are provided also.

```
CONTROL DATA (More Information found in ATLAS User`s guide)

  - Problem Title

    EXAMPLE 1

  - Structural Parameters :

    Number of Nodes   =       26
    Number of Cables  =        2
    Roadway Clearance =   265.12

  - Wind Data :

    Wind Speed (Miles per Hour)              =    60.00
    Wind Direction (Angle from +ve X axis) =     90.0

  - Nonlinear iteration Parameters :

    Number of Iterations (Shape Finder)           =        500
    Number of Iterations (Gravity Solution)       =        500
    Number of Iterations (Wind Solution)          =        500
    Number of Loops for Shape Calculation         =          5
    Number of Cycles (Shape-Stiffness Iteration) =        500
    Force Tolerance for Gravity Solution (%)      =       5.00
    Force Tolerance for Wind Solution (%)         =       3.00
    Pole Displacent Tolerance                     =   0.001000


ECHO OF NODAL POINT INPUT DATA
```

Nodal Point Coordinates				Boundary Conditions					
Node	X	Y	Z	Tx	Ty	Tz	Rx	Ry	Rz
1	600.000	600.000	0.000	F	F	F	F	F	F
2	600.000	600.000	253.120	R	R	R	R	R	R
3	600.000	600.000	341.488	R	R	R	R	R	R
4	608.485	608.485	0.000	F	F	F	F	F	F
5	1680.000	1680.000	0.000	F	F	F	F	F	F
6	1680.000	1680.000	253.120	R	R	R	R	R	R
7	1680.000	1680.000	341.488	R	R	R	R	R	R
8	1688.485	1688.485	0.000	F	F	F	F	F	F
9	827.783	827.783	280.481	R	R	R	R	R	R
10	827.783	827.783	253.120	R	R	R	R	R	R
11	827.783	827.783	225.060	R	R	R	R	R	R
12	897.157	897.157	268.380	R	R	R	R	R	R
13	897.157	897.157	253.120	R	R	R	R	R	R
14	897.157	897.157	238.120	R	R	R	R	R	R
15	1078.025	1078.025	251.054	R	R	R	R	R	R
16	1078.025	1078.025	253.120	R	R	R	R	R	R
17	1078.025	1078.025	225.060	R	R	R	R	R	R
18	1313.402	1313.402	259.296	R	R	R	R	R	R
19	1313.402	1313.402	253.120	R	R	R	R	R	R
20	1313.402	1313.402	225.060	R	R	R	R	R	R
21	1462.061	1462.061	282.444	R	R	R	R	R	R
22	1462.061	1462.061	253.120	R	R	R	R	R	R

23	1462.061	1462.061	238.120	R	R	R	R	R	R
24	1509.136	1509.136	292.669	R	R	R	R	R	R
25	1509.136	1509.136	253.120	R	R	R	R	R	R
26	1509.136	1509.136	225.060	R	R	R	R	R	R

ECHO OF ELEMENT INPUT DATA

1. Pole/Beam Element Data

Number of Property Lines = 1

Property Line = 1
Pole type = NVII
Concrete Strength, F`c (psi) = 6000.00

NOTE : The properties used in the analysis were obtained at the effective heights of the poles and are provided below. For more information refer to the report that accompanies the program.

Pole/Beam Connectivity and Properties Used

	Nodes				Properties					
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	1	2	4	1	214.59	4415.20	7631.09	7631.09	15262.18	1698.15
2	2	3	4	1	173.28	4415.20	3945.37	3945.37	7890.75	1698.15
3	5	6	8	1	214.59	4415.20	7631.09	7631.09	15262.18	1698.15
4	6	7	8	1	173.28	4415.20	3945.37	3945.37	7890.75	1698.15

2. Primary Cable Element Data

Number of Property Lines = 1

Primary Cable Connectivity and Properties

	Nodes				Properties	
Mem	I	J	Mat	Cable	Area	E
1	3	9	1	1	0.0790	27500.0
2	9	12	1	1	0.0790	27500.0
3	12	15	1	1	0.0790	27500.0
4	15	18	1	1	0.0790	27500.0
5	18	21	1	1	0.0790	27500.0
6	21	24	1	1	0.0790	27500.0
7	24	7	1	1	0.0790	27500.0

3. Secondary Cable Element Data

Number of Property Lines = 1

Secondary Cable Connectivity and Properties

	Nodes				Properties	
Mem	I	J	Mat	Cable	Area	E
1	2	10	1	2	0.1500	27500.0
2	10	13	1	2	0.1500	27500.0
3	13	16	1	2	0.1500	27500.0
4	16	19	1	2	0.1500	27500.0
5	19	22	1	2	0.1500	27500.0
6	22	25	1	2	0.1500	27500.0
7	25	6	1	2	0.1500	27500.0

4. Connector Element Data

Number of Property Lines = 2

Connector Connectivity and Properties

Mem	Nodes I	J	K	Mat	Area	Properties E	I33	I22	J	G
1	9	10	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
2	12	13	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	15	16	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
4	18	19	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
5	21	22	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
6	24	25	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8

5. Light Element Data

Number of Property Lines = 2

Property Line = 1
Projected area on X-Z plane = 591.36
Projected area on Y-Z plane = 591.36

Property Line = 2
Projected area on X-Z plane = 720.00
Projected area on Y-Z plane = 0.00

Light Connectivity and Properties

Mem	Nodes I	J	K	Mat	Area	Properties E	I33	I22	J	G
1	10	11	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
2	13	14	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	16	17	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
4	19	20	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8
5	22	23	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
6	25	26	1	1	1.0700	29000.0	0.6660	0.6660	1.3300	11153.8

CONCENTRATED APPLIED LOADS

- Sign weights

Node	X	Y	Z
3	0.00000	0.00000	-0.00364
7	0.00000	0.00000	-0.00273
9	0.00000	0.00000	-0.00892
10	0.00000	0.00000	-0.00892
11	0.00000	0.00000	-0.16325
12	0.00000	0.00000	-0.00583
13	0.00000	0.00000	-0.00583
14	0.00000	0.00000	-0.00399
15	0.00000	0.00000	-0.00665
16	0.00000	0.00000	-0.00665
17	0.00000	0.00000	-0.16512
18	0.00000	0.00000	-0.00729
19	0.00000	0.00000	-0.00729
20	0.00000	0.00000	-0.16461
21	0.00000	0.00000	-0.00662
22	0.00000	0.00000	-0.00662
23	0.00000	0.00000	-0.00312
24	0.00000	0.00000	-0.00986
25	0.00000	0.00000	-0.00986
26	0.00000	0.00000	-0.16198

The following sections present the results of the analysis. The results consist of the solution for the gravity loads and the solution when all the loads (gravity and wind)

are applied on the structure at the same time. The results of the gravity solution consist of the final cooordinates of the structure and the nodal point displacements. Also provided are the different element forces which are discussed below.

```
*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
|                                                     |
|   G R A V I T Y   S O L U T I O N   R E S U L T S   |
|                                                     |
*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
```

Final Coordinates

Node	X	Y	Z
1	600.0001	600.0001	0.0000
2	600.4889	600.4889	253.1199
3	600.7876	600.7876	341.4874
4	608.4854	608.4854	0.0000
5	1680.0000	1680.0000	0.0000
6	1679.5112	1679.5112	253.1199
7	1679.2125	1679.2125	341.4873
8	1688.4850	1688.4850	0.0000
9	827.6932	827.6932	290.2114
10	827.6932	827.6932	253.1309
11	827.6932	827.6932	225.0708
12	897.2593	897.2593	282.7340
13	897.2593	897.2593	253.1309
14	897.2593	897.2593	238.1309
15	1078.1576	1078.1576	265.1303
16	1078.1576	1078.1576	253.1302
17	1078.1576	1078.1576	225.0701
18	1313.8479	1313.8479	269.5403
19	1313.8479	1313.8479	253.1268
20	1313.8479	1313.8479	225.0667
21	1462.5416	1462.5416	289.6515
22	1462.5416	1462.5416	253.1265
23	1462.5416	1462.5416	238.1265
24	1509.7526	1509.7526	296.5392
25	1509.7526	1509.7526	253.1262
26	1509.7526	1509.7526	225.0660

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.4888	0.4888	-0.0001	-0.0032	0.0032	0.0000
3	0.7875	0.7875	-0.0001	-0.0035	0.0035	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.4888	-0.4888	-0.0001	0.0032	-0.0032	0.0000
7	-0.7875	-0.7875	-0.0002	0.0035	-0.0035	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	0.0000	0.0000	0.0111	0.0000	0.0000	0.0000
10	0.0000	0.0000	0.0109	0.0000	0.0000	0.0000
11	0.0000	0.0000	0.0108	0.0000	0.0000	0.0000
12	0.0000	0.0000	0.0109	0.0000	0.0000	0.0000
13	0.0000	0.0000	0.0109	0.0000	0.0000	0.0000
14	0.0000	0.0000	0.0109	0.0000	0.0000	0.0000
15	0.0000	0.0000	0.0103	0.0000	0.0000	0.0000
16	0.0000	0.0000	0.0102	0.0000	0.0000	0.0000
17	0.0000	0.0000	0.0101	0.0000	0.0000	0.0000
18	0.0000	0.0000	0.0069	0.0000	0.0000	0.0000

19	0.0000	0.0000	0.0068	0.0000	0.0000	0.0000
20	0.0000	0.0000	0.0067	0.0000	0.0000	0.0000
21	0.0000	0.0000	0.0065	0.0000	0.0000	0.0000
22	0.0000	0.0000	0.0065	0.0000	0.0000	0.0000
23	0.0000	0.0000	0.0065	0.0000	0.0000	0.0000
24	0.0000	0.0000	0.0064	0.0000	0.0000	0.0000
25	0.0000	0.0000	0.0062	0.0000	0.0000	0.0000
26	0.0000	0.0000	0.0060	0.0000	0.0000	0.0000

The frame member forces consist of moments, axial and shear forces. The forces are expressed in the element local system which is defined implicitly in the input data file when the third (K-node) is provided. ATLAS considers the Xm - Ym - Zm coordinate system as the element local system. The results for the individual frame elements are provided in the local system. ATLAS considers the Xm axis as the axis along the element chord and the Xm - Ym plane as the strong plane of the element.

```
- Frame Member Forces

  Member #   1
                              Node I        Node J

  Axial Force      =          0.3500       -0.3500
  Shear Xm - Ym    =         -3.1733        3.1733
  Shear Xm - Zm    =          0.0000        0.0000
  Torsion          =          0.0000        0.0000
  Moment About Ym =           0.0000        0.0000
  Moment About Zm =        -994.8117      191.5849

  Member #   2
                              Node I        Node J

  Axial Force      =          0.3501       -0.3501
  Shear Xm - Ym    =         -2.1680        2.1680
  Shear Xm - Zm    =          0.0000        0.0000
  Torsion          =          0.0000        0.0000
  Moment About Ym =           0.0000        0.0000
  Moment About Zm =        -191.5849        0.0000

  Member #   3
                              Node I        Node J

  Axial Force      =          0.4087       -0.4087
  Shear Xm - Ym    =          3.1731       -3.1731
  Shear Xm - Zm    =          0.0000        0.0000
  Torsion          =          0.0000        0.0000
  Moment About Ym =           0.0000        0.0000
  Moment About Zm =         994.7988     -191.6316

  Member #   4
                              Node I        Node J

  Axial Force      =          0.4095       -0.4095
  Shear Xm - Ym    =          2.1686       -2.1686
  Shear Xm - Zm    =          0.0000        0.0000
  Torsion          =          0.0000        0.0000
  Moment About Ym =           0.0000        0.0000
  Moment About Zm =         191.6316        0.0000
```

The cable element tensions and stresses are provided for each individual cable segment. Therefore, ATLAS provides the tension and stress in each primary and secondary cable as they are defined in the input data file. Also provided are the forces on the poles due to the cable tensions. These forces are necessary for the design of the foundations.

- Primary (Catenary) Cable Forces   Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	2.2021	27.8745	3	1.5376	1.5376	-0.3475
2	2.1809	27.6069				
3	2.1798	27.5926				
4	2.1749	27.5299				
5	2.1846	27.6531				
6	2.1862	27.6736				
7	2.2123	28.0032	7	-1.5375	-1.5375	-0.4078

- Secondary (Messenger) Cable Forces   Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.0000	6.6667	2	0.7071	0.7071	0.0000
2	1.0000	6.6667				
3	1.0000	6.6667				
4	1.0000	6.6667				
5	1.0000	6.6667				
6	1.0000	6.6667				
7	1.0000	6.6667	6	-0.7071	-0.7071	0.0000

The light and hanger forces follow the same pattern as those of the frame elements which are discussed previously.

- Light Member Forces

Member # 1

		Node I	Node J
Axial Force	=	-0.1633	0.1633
Shear Xm - Ym	=	0.0000	0.0000
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	0.0000	0.0000

Member # 2

		Node I	Node J
Axial Force	=	-0.0040	0.0040
Shear Xm - Ym	=	0.0000	0.0000
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	0.0000	0.0000

```
Member #   3
                            Node I          Node J

Axial Force      =         -0.1651          0.1651
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

Member #   4
                            Node I          Node J

Axial Force      =         -0.1646          0.1646
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

Member #   5
                            Node I          Node J

Axial Force      =         -0.0031          0.0031
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

Member #   6
                            Node I          Node J

Axial Force      =         -0.1620          0.1620
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

- Hanger (Connector) Member Forces

Member #   1
                            Node I          Node J

Axial Force      =         -0.1722          0.1722
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

Member #   2
                            Node I          Node J

Axial Force      =         -0.0098          0.0098
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
Moment About Ym =           0.0000          0.0000
Moment About Zm =           0.0000          0.0000

Member #   3
                            Node I          Node J

Axial Force      =         -0.1718          0.1718
Shear Xm - Ym    =          0.0000          0.0000
Shear Xm - Zm    =          0.0000          0.0000
Torsion          =          0.0000          0.0000
```

```
Moment About Ym =        0.0000         0.0000
Moment About Zm =        0.0000         0.0000

Member #   4
                         Node I         Node J

Axial Force     =       -0.1719         0.1719
Shear Xm - Ym   =        0.0000         0.0000
Shear Xm - Zm   =        0.0000         0.0000
Torsion         =        0.0000         0.0000
Moment About Ym =        0.0000         0.0000
Moment About Zm =        0.0000         0.0000

Member #   5
                         Node I         Node J

Axial Force     =       -0.0097         0.0097
Shear Xm - Ym   =        0.0000         0.0000
Shear Xm - Zm   =        0.0000         0.0000
Torsion         =        0.0000         0.0000
Moment About Ym =        0.0000         0.0000
Moment About Zm =        0.0000         0.0000

Member #   6
                         Node I         Node J

Axial Force     =       -0.1726         0.1726
Shear Xm - Ym   =        0.0000         0.0000
Shear Xm - Zm   =        0.0000         0.0000
Torsion         =        0.0000         0.0000
Moment About Ym =        0.0000         0.0000
Moment About Zm =        0.0000         0.0000
```

The following section provides the results for the analysis of the structure when all the loads are applied on it. The presentation of the results is the same as the one previously discussed in the gravity solution. The additional information that is provided in this section is the information regarding the hanger and essentially the signal light/sign element rotations. These rotations are used for the determination of the drag force and uplift on the different elements. ATLAS provides the rotation for each element individually. The wind load that is acting on each of the individual element is based on the particular element rotation.

```
*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*
|                                                     |
|      W I N D   S O L U T I O N   R E S U L T S      |
|                                                     |
*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*-*

Final Coordinates

   Node            X              Y              Z

     1        600.0001       600.0001         0.0000
     2        600.5999       600.6429       253.1199
     3        600.9455       601.0122       341.4874
     4        608.4854       608.4854         0.0000
     5       1680.0000      1680.0000         0.0000
```

6	1679.3747	1679.4235	253.1199
7	1679.0072	1679.0835	341.4874
8	1688.4850	1688.4850	0.0000
9	823.3460	832.0215	289.3163
10	816.8512	838.5104	253.3904
11	811.9624	843.4772	226.2095
12	892.1199	902.3284	281.7387
13	884.2034	910.2775	254.3437
14	880.1945	914.3361	240.4708
15	1068.0201	1088.1113	265.1949
16	1064.8603	1091.5755	254.1488
17	1057.4969	1099.7023	228.3204
18	1304.6038	1322.8121	270.5013
19	1300.6589	1327.3423	255.2265
20	1293.9416	1335.1180	229.1154
21	1457.5088	1467.1497	290.5135
22	1450.0878	1475.4029	255.7158
23	1447.0417	1478.8292	241.4335
24	1505.4960	1513.5511	297.4705
25	1499.1484	1520.7180	255.1262
26	1495.0722	1525.4173	227.7644

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.5998	0.6428	-0.0001	-0.0040	0.0037	0.0000
3	0.9454	1.0121	-0.0001	-0.0043	0.0040	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.6253	-0.5765	-0.0001	0.0036	-0.0039	0.0000
7	-0.9928	-0.9165	-0.0001	0.0040	-0.0043	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	-4.3472	4.3283	-0.8840	0.1781	0.1791	0.0000
10	-10.8420	10.8172	0.2704	0.1798	0.1784	-0.0004
11	-15.7308	15.7840	1.1495	0.1810	0.1779	-0.0008
12	-5.1394	5.0691	-0.9845	0.2817	0.2813	0.0000
13	-13.0560	13.0182	1.2237	0.2837	0.2815	-0.0006
14	-17.0648	17.0768	2.3508	0.2847	0.2815	-0.0008
15	-10.1375	9.9537	0.0749	0.3038	0.2788	0.0000
16	-13.2973	13.4179	1.0288	0.3041	0.2784	-0.0002
17	-20.6607	21.5447	3.2604	0.3049	0.2776	-0.0007
18	-9.2441	8.9642	0.9679	0.2881	0.2529	0.0000
19	-13.1890	13.4944	2.1065	0.2886	0.2525	-0.0003
20	-19.9063	21.2701	4.0554	0.2896	0.2517	-0.0007
21	-5.0328	4.6081	0.8686	0.2320	0.2101	0.0000
22	-12.4538	12.8613	2.5958	0.2346	0.2103	-0.0006
23	-15.4998	16.2876	3.3135	0.2356	0.2103	-0.0008
24	-4.2566	3.7985	0.9377	0.1669	0.1491	0.0000
25	-10.6042	10.9654	2.0062	0.1691	0.1483	-0.0005
26	-14.6804	15.6647	2.7044	0.1704	0.1478	-0.0008

- Frame Member Forces

Member # 1

		Node I	Node J
Axial Force	=	0.2665	-0.2665
Shear Xm - Ym	=	-4.6221	4.6221
Shear Xm - Zm	=	-0.1734	0.1734
Torsion	=	0.0000	0.0000
Moment About Ym	=	46.6146	-2.7361
Moment About Zm	=	-1314.0531	144.1079

Member # 2

		Node I	Node J
Axial Force	=	0.2690	-0.2690
Shear Xm - Ym	=	-1.6308	1.6308
Shear Xm - Zm	=	-0.0310	0.0310
Torsion	=	0.0000	0.0000
Moment About Ym	=	2.7361	0.0000
Moment About Zm	=	-144.1079	0.0000

Member # 3

		Node I	Node J
Axial Force	=	0.3482	-0.3482
Shear Xm - Ym	=	4.2571	-4.2571
Shear Xm - Zm	=	-0.1910	0.1910
Torsion	=	0.0000	0.0000
Moment About Ym	=	52.4696	-4.1279
Moment About Zm	=	1252.9627	-175.4061

Member # 4

		Node I	Node J
Axial Force	=	0.3672	-0.3672
Shear Xm - Ym	=	1.9850	-1.9850
Shear Xm - Zm	=	-0.0467	0.0467
Torsion	=	0.0000	0.0000
Moment About Ym	=	4.1279	0.0000
Moment About Zm	=	175.4061	0.0000

- Primary (Catenary) Cable Forces — Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.6530	20.9243	3	1.1316	1.1754	-0.2654
2	1.6839	21.3152				
3	1.7119	21.6696				
4	1.8351	23.2287				
5	1.9332	24.4715				
6	1.9589	24.7956				
7	2.0194	25.5619	7	-1.4371	-1.3710	-0.3646

- Secondary (Messenger) Cable Forces — Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	2.9953	19.9688	2	2.0149	2.2163	0.0025
2	2.8825	19.2167				
3	2.7937	18.6249				
4	2.6075	17.3834				
5	2.4593	16.3956				
6	2.3768	15.8452				
7	2.2778	15.1850	6	-1.7094	-1.5053	0.0190

- Light Member Forces

Member # 1
Rotation Angle in X-Z Plane (Degrees) = 12.13
Rotation Angle in Y-Z Plane (Degrees) = 11.27

		Node I	Node J
Axial Force	=	-0.1552	0.1552
Shear Xm - Ym	=	0.0638	-0.0638
Shear Xm - Zm	=	-0.0257	0.0257
Torsion	=	0.0001	-0.0001

```
Moment About Ym =       0.7208        -0.0005
Moment About Zm =       1.7905        -0.0004

Member #   2
Rotation Angle in X-Z Plane (Degrees) =   17.85
Rotation Angle in Y-Z Plane (Degrees) =   18.11

                        Node I         Node J

Axial Force     =      -0.0019         0.0019
Shear Xm - Ym   =       0.0581        -0.0581
Shear Xm - Zm   =      -0.0628         0.0628
Torsion         =       0.0001        -0.0001
Moment About Ym =       0.9400         0.0015
Moment About Zm =       0.8733        -0.0025

Member #   3
Rotation Angle in X-Z Plane (Degrees) =   18.50
Rotation Angle in Y-Z Plane (Degrees) =   17.14

                        Node I         Node J

Axial Force     =      -0.1582         0.1582
Shear Xm - Ym   =       0.0616        -0.0616
Shear Xm - Zm   =      -0.0013         0.0013
Torsion         =       0.0000         0.0000
Moment About Ym =       0.0372        -0.0010
Moment About Zm =       1.7309        -0.0024

Member #   4
Rotation Angle in X-Z Plane (Degrees) =   18.30
Rotation Angle in Y-Z Plane (Degrees) =   16.47

                        Node I         Node J

Axial Force     =      -0.1568         0.1568
Shear Xm - Ym   =       0.0629        -0.0629
Shear Xm - Zm   =      -0.0057         0.0057
Torsion         =       0.0000         0.0000
Moment About Ym =       0.1604        -0.0009
Moment About Zm =       1.7649        -0.0006

Member #   5
Rotation Angle in X-Z Plane (Degrees) =   15.03
Rotation Angle in Y-Z Plane (Degrees) =   13.67

                        Node I         Node J

Axial Force     =      -0.0001         0.0001
Shear Xm - Ym   =       0.0600        -0.0600
Shear Xm - Zm   =      -0.0636         0.0636
Torsion         =       0.0001        -0.0001
Moment About Ym =       0.9546        -0.0004
Moment About Zm =       0.8998         0.0000

Member #   6
Rotation Angle in X-Z Plane (Degrees) =   11.19
Rotation Angle in Y-Z Plane (Degrees) =   10.51

                        Node I         Node J

Axial Force     =      -0.1543         0.1543
Shear Xm - Ym   =       0.0664        -0.0664
Shear Xm - Zm   =      -0.0307         0.0307
Torsion         =       0.0001        -0.0001
Moment About Ym =       0.8634        -0.0006
Moment About Zm =       1.8643         0.0002
```

- Hanger (Connector) Member Forces

Member # 1

		Node I	Node J
Axial Force	=	-0.1623	0.1623
Shear Xm - Ym	=	-0.0483	0.0483
Shear Xm - Zm	=	0.0195	-0.0195
Torsion	=	-0.0029	0.0029
Moment About Ym	=	-0.0001	-0.7219
Moment About Zm	=	0.0008	-1.7907

Member # 2

		Node I	Node J
Axial Force	=	-0.0016	0.0016
Shear Xm - Ym	=	-0.0295	0.0295
Shear Xm - Zm	=	0.0317	-0.0317
Torsion	=	-0.0027	0.0027
Moment About Ym	=	-0.0001	-0.9386
Moment About Zm	=	0.0011	-0.8757

Member # 3

		Node I	Node J
Axial Force	=	-0.1697	0.1697
Shear Xm - Ym	=	-0.1445	0.1445
Shear Xm - Zm	=	0.0032	-0.0032
Torsion	=	-0.0001	0.0001
Moment About Ym	=	-0.0002	-0.0384
Moment About Zm	=	-0.0004	-1.7337

Member # 4

		Node I	Node J
Axial Force	=	-0.1858	0.1858
Shear Xm - Ym	=	-0.1076	0.1076
Shear Xm - Zm	=	0.0099	-0.0099
Torsion	=	-0.0005	0.0005
Moment About Ym	=	-0.0002	-0.1616
Moment About Zm	=	0.0001	-1.7656

Member # 5

		Node I	Node J
Axial Force	=	-0.0058	0.0058
Shear Xm - Ym	=	-0.0246	0.0246
Shear Xm - Zm	=	0.0262	-0.0262
Torsion	=	-0.0033	0.0033
Moment About Ym	=	0.0000	-0.9553
Moment About Zm	=	0.0011	-0.8996

Member # 6

		Node I	Node J
Axial Force	=	-0.1870	0.1870
Shear Xm - Ym	=	-0.0429	0.0429
Shear Xm - Zm	=	0.0199	-0.0199
Torsion	=	-0.0040	0.0040
Moment About Ym	=	0.0000	-0.8648
Moment About Zm	=	0.0009	-1.8637

Example 6.2

The following is a three dimensional L-shaped structure as shown on Figure 6.13 below. The required clearance is 17 *ft*. The catenary cable has 1/4 *in* diameter and the messenger cable has 3/8 *in* diameter. The poles are made of prestress concrete and they are both of type *NVI*. The signal lights and sign arrangement as well as their type are also shown on Figure 6.13. The input as well as the output data files that are involved in the analysis of this particular system are also shown below.

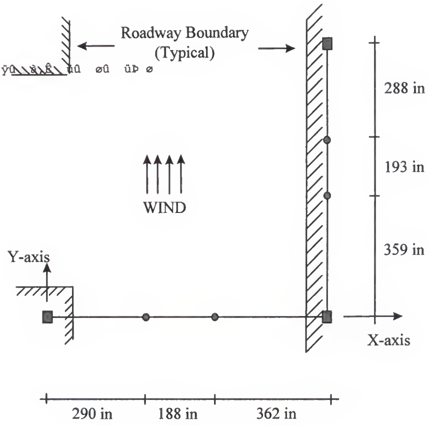

Figure 6. 13 Structure for Example 6.2

```
CONTROL
Title= Example 2 - Three Pole Configuration
CLEAR= 265.12
CABLE=  4
SPEED=  50.0
ANGLE= 90
STATUS=3
:
CABLES
  1  3  7    S=     6.0    W=        0.23E-04    P=0
  2  2  6    T=     1.0    W=        0.43E-04    P=1
  3  7 11    S=     6.0    W=        0.23E-04    P=0
  4  6 10    T=     1.0    W=        0.43E-04    P=1
:
COORDINATE
     1 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.0000000E+00
     2 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.2531200E+03
     3 X= 0.6000001E+03 Y= 0.6000001E+03 Z= 0.3071200E+03
     4 X= 0.6120001E+03 Y= 0.6000001E+03 Z= 0.0000000E+00
     5 X= 0.1440000E+04 Y= 0.6000001E+03 Z= 0.0000000E+00
     6 X= 0.1440000E+04 Y= 0.6000001E+03 Z= 0.2531200E+03
     7 X= 0.1440000E+04 Y= 0.6000001E+03 Z= 0.3071200E+03
     8 X= 0.1450392E+04 Y= 0.6060001E+03 Z= 0.0000000E+00
     9 X= 0.1440000E+04 Y= 0.1440000E+04 Z= 0.0000000E+00
    10 X= 0.1440000E+04 Y= 0.1440000E+04 Z= 0.2531200E+03
    11 X= 0.1440000E+04 Y= 0.1440000E+04 Z= 0.3071200E+03
    12 X= 0.1450392E+04 Y= 0.1446000E+04 Z= 0.0000000E+00
    13 X= 0.8897240E+03 Y= 0.6000001E+03 C=    1
    14 X= 0.8897240E+03 Y= 0.6000001E+03 Z= 0.2531200E+03
    15 X= 0.8897240E+03 Y= 0.6000001E+03 Z= 0.2250600E+03
    16 X= 0.1078025E+04 Y= 0.6000001E+03 C=    1
    17 X= 0.1078025E+04 Y= 0.6000001E+03 Z= 0.2531200E+03
    18 X= 0.1078025E+04 Y= 0.6000001E+03 Z= 0.2250600E+03
    19 X= 0.1440000E+04 Y= 0.9590982E+03 C=    3
    20 X= 0.1440000E+04 Y= 0.9590982E+03 Z= 0.2531200E+03
    21 X= 0.1440000E+04 Y= 0.9590982E+03 Z= 0.2250600E+03
    22 X= 0.1440000E+04 Y= 0.1152355E+04 C=    3
    23 X= 0.1440000E+04 Y= 0.1152355E+04 Z= 0.2531200E+03
    24 X= 0.1440000E+04 Y= 0.1152355E+04 Z= 0.2250600E+03
:
BOUNDARY
     1 DOF=f f f f f f
     2 DOF=r r r r r r
     3 DOF=r r r r r r
     4 DOF=f f f f f f
     5 DOF=f f f f f f
     6 DOF=r r r r r r
     7 DOF=r r r r r r
     8 DOF=f f f f f f
     9 DOF=f f f f f f
    10 DOF=r r r r r r
    11 DOF=r r r r r r
    12 DOF=f f f f f f
    13 DOF=r r r r r r
    14 DOF=r r r r r r
    15 DOF=r r r r r r
    16 DOF=r r r r r r
    17 DOF=r r r r r r
    18 DOF=r r r r r r
    19 DOF=r r r r r r
    20 DOF=r r r r r r
    21 DOF=r r r r r r
    22 DOF=r r r r r r
    23 DOF=r r r r r r
    24 DOF=r r r r r r
:
```

```
PRIMARY
    6,    1
  1 A=       0.0790 E=  0.27500E+05
   1     3,  13 M=  1 C=  1
   2    13,  16 M=  1 C=  1
   3    16,   7 M=  1 C=  1
   4     7,  19 M=  1 C=  3
   5    19,  22 M=  1 C=  3
   6    22,  11 M=  1 C=  3
:
SECONDARY
    6,    1
  1 A=       0.1500 E=  0.27500E+05
   1     2,  14 M=  1 C=  2
   2    14,  17 M=  1 C=  2
   3    17,   6 M=  1 C=  2
   4     6,  20 M=  1 C=  4
   5    20,  23 M=  1 C=  4
   6    23,  10 M=  1 C=  4
:
CONNECTORS
    4,    1
  1 A=       0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00
   1    13,  14,   1 M=  1
   2    16,  17,   1 M=  1
   3    19,  20,   5 M=  1
   4    22,  23,   5 M=  1
:
LIGHTS
    4,    2
  1 A=       0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00 S= 1 P=  0.12552E+04, 0.48000E+03
  2 A=       0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00 S= 1 P=  0.48000E+03, 0.12552E+04
   1    14,  15,   1 M=  1
   2    17,  18,   1 M=  1
   3    20,  21,   5 M=  2
   4    23,  24,   5 M=  2
:
BEAM
    6,    1
  1  T= NVII   FC=      6000.0
   1     1,   2,   4 M=  1
   2     2,   3,   4 M=  1
   3     5,   6,   8 M=  1
   4     6,   7,   8 M=  1
   5     9,  10,  12 M=  1
   6    10,  11,  12 M=  1
:
SIGNS
   13 F=  0.00000E+00, 0.00000E+00,-0.10721E+00
   16 F=  0.00000E+00, 0.00000E+00,-0.10759E+00
   19 F=  0.00000E+00, 0.00000E+00,-0.10762E+00
   22 F=  0.00000E+00, 0.00000E+00,-0.10726E+00
:
WIND
:

LOADS
    3 F=  0.00000E+00, 0.00000E+00,-0.32695E-02
    7 F=  0.00000E+00, 0.00000E+00,-0.81311E-02
   11 F=  0.00000E+00, 0.00000E+00,-0.32461E-02
   13 F=  0.00000E+00, 0.00000E+00,-0.35128E-02
   14 F=  0.00000E+00, 0.00000E+00,-0.35128E-02
   15 F=  0.00000E+00, 0.00000E+00,-0.10019E+00
   16 F=  0.00000E+00, 0.00000E+00,-0.32941E-02
   17 F=  0.00000E+00, 0.00000E+00,-0.32941E-02
   18 F=  0.00000E+00, 0.00000E+00,-0.10100E+00
```

```
19 F=  0.00000E+00, 0.00000E+00,-0.32996E-02
20 F=  0.00000E+00, 0.00000E+00,-0.32996E-02
21 F=  0.00000E+00, 0.00000E+00,-0.10102E+00
22 F=  0.00000E+00, 0.00000E+00,-0.35215E-02
23 F=  0.00000E+00, 0.00000E+00,-0.35215E-02
24 F=  0.00000E+00, 0.00000E+00,-0.10022E+00
```

```
**********************************************************************
|                                                                    |
|      ##         #####       #            ##        ####            |
|     #  #          #         #           #  #      #                |
|    #    #         #         #          #    #      ####           |
|    ######         #         #          ######          #          |
|    #    #         #         #          #    #     #    #          |
|    #    #         #         ######     #    #      ####           |
|                                                                    |
|             Analysis of Traffic Lights And Signs                  |
|                                                                    |
|                          Version 4.0                              |
|                                                                    |
|          Developed by :                                           |
|                                                                    |
|          Dr Marc I. Hoit              Dr Ronald A. Cook           |
|          Mr Petros M. Christou        Ms Adeola K. Adediran       |
|                                                                    |
|               Department of Civil Engineering                      |
|                    University of Florida                          |
|                    Gainesville, Fl 32608                          |
|                                                                    |
**********************************************************************
```

```
Input Data File = lshape
Output Data File = lshp.out


CONTROL DATA (More Information found in ATLAS User`s guide)

 - Problem Title

   THREE POLE STUFF

 - Structural Parameters :

   Number of Nodes    =        24
   Number of Cables   =         4
   Roadway Clearance  =    265.12

 - Wind Data :

   Wind Speed (Miles per Hour)             =    50.00
   Wind Direction (Angle from +ve X axis)  =     90.0

 - Nonlinear iteration Parameters :

   Number of Iterations (Shape Finder)           =        500
   Number of Iterations (Gravity Solution)       =        500
   Number of Iterations (Wind Solution)          =        500
   Number of Loops for Shape Calculation         =          5
   Number of Cycles (Shape-Stiffness Iteration)  =        500
   Force Tolerance for Gravity Solution (%)      =       5.00
   Force Tolerance for Wind Solution (%)         =       3.00
   Pole Displacent Tolerance                     =   0.001000
```

ECHO OF NODAL POINT INPUT DATA

Nodal Point Coordinates				Boundary Conditions					
Node	X	Y	Z	Tx	Ty	Tz	Rx	Ry	Rz
1	600.000	600.000	0.000	F	F	F	F	F	F
2	600.000	600.000	253.120	R	R	R	R	R	R
3	600.000	600.000	307.120	R	R	R	R	R	R
4	612.000	600.000	0.000	F	F	F	F	F	F
5	1440.000	600.000	0.000	F	F	F	F	F	F
6	1440.000	600.000	253.120	R	R	R	R	R	R
7	1440.000	600.000	307.120	R	R	R	R	R	R
8	1450.392	606.000	0.000	F	F	F	F	F	F
9	1440.000	1440.000	0.000	F	F	F	F	F	F
10	1440.000	1440.000	253.120	R	R	R	R	R	R
11	1440.000	1440.000	307.120	R	R	R	R	R	R
12	1450.392	1446.000	0.000	F	F	F	F	F	F
13	889.724	600.000	261.569	R	R	R	R	R	R
14	889.724	600.000	253.120	R	R	R	R	R	R
15	889.724	600.000	225.060	R	R	R	R	R	R
16	1078.025	600.000	257.682	R	R	R	R	R	R
17	1078.025	600.000	253.120	R	R	R	R	R	R
18	1078.025	600.000	225.060	R	R	R	R	R	R
19	1440.000	959.098	257.780	R	R	R	R	R	R
20	1440.000	959.098	253.120	R	R	R	R	R	R
21	1440.000	959.098	225.060	R	R	R	R	R	R
22	1440.000	1152.355	261.725	R	R	R	R	R	R
23	1440.000	1152.355	253.120	R	R	R	R	R	R
24	1440.000	1152.355	225.060	R	R	R	R	R	R

ECHO OF ELEMENT INPUT DATA

1. Pole/Beam Element Data

Number of Property Lines = 1

Property Line = 1
Pole type = NVII
Concrete Strength, F`c (psi) = 6000.00

NOTE : The properties used in the analysis were obtained at the effective heights of the poles and are provided below. For more information refer to the report that accompanies the program.

Pole/Beam Connectivity and Properties Used

	Nodes					Properties				
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	1	2	4	1	205.78	4415.20	6895.18	6895.18	13790.36	1698.15
2	2	3	4	1	169.25	4415.20	3607.12	3607.12	7214.24	1698.15
3	5	6	8	1	205.78	4415.20	6895.18	6895.18	13790.36	1698.15
4	6	7	8	1	169.25	4415.20	3607.12	3607.12	7214.24	1698.15
5	9	10	12	1	205.78	4415.20	6895.18	6895.18	13790.36	1698.15
6	10	11	12	1	169.25	4415.20	3607.12	3607.12	7214.24	1698.15

2. Primary Cable Element Data

Number of Property Lines = 1

Primary Cable Connectivity and Properties

	Nodes				Properties	
Mem	I	J	Mat	Cable	Area	E
1	3	13	1	1	0.0790	27500.0

2	13	16	1	1	0.0790	27500.0
3	16	7	1	1	0.0790	27500.0
4	7	19	1	3	0.0790	27500.0
5	19	22	1	3	0.0790	27500.0
6	22	11	1	3	0.0790	27500.0

3. Secondary Cable Element Data

Number of Property Lines = 1

Secondary Cable Connectivity and Properties

	Nodes				Properties	
Mem	I	J	Mat	Cable	Area	E
1	2	14	1	2	0.1500	27500.0
2	14	17	1	2	0.1500	27500.0
3	17	6	1	2	0.1500	27500.0
4	6	20	1	4	0.1500	27500.0
5	20	23	1	4	0.1500	27500.0
6	23	10	1	4	0.1500	27500.0

4. Connector Element Data

Number of Property Lines = 1

Connector Connectivity and Properties

	Nodes				Properties					
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	13	14	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
2	16	17	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	19	20	5	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
4	22	23	5	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8

5. Light Element Data

Number of Property Lines = 2

Property Line = 1
Projected area on X-Z plane = 1255.20
Projected area on Y-Z plane = 480.00

Property Line = 2
Projected area on X-Z plane = 1255.20
Projected area on Y-Z plane = 480.00

Light Connectivity and Properties

	Nodes				Properties					
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	14	15	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
2	17	18	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	20	21	5	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
4	23	24	5	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8

CONCENTRATED APPLIED LOADS

- Sign weights

Node	X	Y	Z
3	0.00000	0.00000	-0.00327
7	0.00000	0.00000	-0.00813
11	0.00000	0.00000	-0.00325
13	0.00000	0.00000	-0.00351

14	0.00000	0.00000	-0.00351
15	0.00000	0.00000	-0.10019
16	0.00000	0.00000	-0.00329
17	0.00000	0.00000	-0.00329
18	0.00000	0.00000	-0.10100
19	0.00000	0.00000	-0.00330
20	0.00000	0.00000	-0.00330
21	0.00000	0.00000	-0.10102
22	0.00000	0.00000	-0.00352
23	0.00000	0.00000	-0.00352
24	0.00000	0.00000	-0.10022

G R A V I T Y   S O L U T I O N   R E S U L T S

Final Coordinates

Node	X	Y	Z
1	600.0001	600.0001	0.0000
2	600.3758	600.0001	253.1200
3	600.5039	600.0001	307.1200
4	612.0001	600.0001	0.0000
5	1440.0000	600.0001	0.0000
6	1439.6247	600.3741	253.1199
7	1439.4967	600.5016	307.1199
8	1450.3920	606.0001	0.0000
9	1440.0000	1440.0000	0.0000
10	1440.0000	1439.6256	253.1200
11	1440.0000	1439.4979	307.1200
12	1450.3920	1446.0000	0.0000
13	889.4520	600.0001	267.2233
14	889.4520	600.0001	253.1217
15	889.4520	600.0001	225.0616
16	1077.8875	600.0001	265.1220
17	1077.8875	600.0001	253.1220
18	1077.8875	600.0001	225.0618
19	1440.0000	959.2212	265.1213
20	1440.0000	959.2212	253.1213
21	1440.0000	959.2212	225.0611
22	1440.0000	1152.6190	267.2668
23	1440.0000	1152.6190	253.1210
24	1440.0000	1152.6190	225.0609

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3757	0.0000	0.0000	0.0000	0.0023	0.0000
3	0.5038	0.0000	0.0000	0.0000	0.0024	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.3753	0.3740	-0.0001	-0.0023	-0.0023	0.0000
7	-0.5033	0.5015	-0.0001	-0.0024	-0.0024	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
10	0.0000	-0.3744	0.0000	0.0023	0.0000	0.0000
11	0.0000	-0.5021	0.0000	0.0024	0.0000	0.0000
12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13	0.0000	0.0000	0.0018	0.0000	0.0000	0.0000
14	0.0000	0.0000	0.0017	0.0000	0.0000	0.0000
15	0.0000	0.0000	0.0016	0.0000	0.0000	0.0000
16	0.0000	0.0000	0.0020	0.0000	0.0000	0.0000

17	0.0000	0.0000	0.0020	0.0000	0.0000	0.0000
18	0.0000	0.0000	0.0018	0.0000	0.0000	0.0000
19	0.0000	0.0000	0.0013	0.0000	0.0000	0.0000
20	0.0000	0.0000	0.0013	0.0000	0.0000	0.0000
21	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000
22	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000
23	0.0000	0.0000	0.0010	0.0000	0.0000	0.0000
24	0.0000	0.0000	0.0009	0.0000	0.0000	0.0000

- Frame Member Forces

Member # 1

		Node I	Node J
Axial Force	=	0.1199	-0.1199
Shear Xm - Ym	=	-1.8454	1.8454
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	-512.7337	45.6231

Member # 2

		Node I	Node J
Axial Force	=	0.1199	-0.1199
Shear Xm - Ym	=	-0.8449	0.8449
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	-45.6231	0.0000

Member # 3

		Node I	Node J
Axial Force	=	0.2046	-0.2046
Shear Xm - Ym	=	0.6774	-0.6774
Shear Xm - Zm	=	-2.5133	2.5133
Torsion	=	0.0000	0.0000
Moment About Ym	=	698.1712	-62.0106
Moment About Zm	=	188.2943	-16.8234

Member # 4

		Node I	Node J
Axial Force	=	0.2046	-0.2046
Shear Xm - Ym	=	0.3115	-0.3115
Shear Xm - Zm	=	-1.1483	1.1483
Torsion	=	0.0000	0.0000
Moment About Ym	=	62.0106	0.0000
Moment About Zm	=	16.8234	0.0000

Member # 5

		Node I	Node J
Axial Force	=	0.1198	-0.1198
Shear Xm - Ym	=	0.9200	-0.9200
Shear Xm - Zm	=	1.5934	-1.5934
Torsion	=	0.0000	0.0000
Moment About Ym	=	-442.5818	39.2691
Moment About Zm	=	255.5322	-22.6727

Member # 6

		Node I	Node J
Axial Force	=	0.1199	-0.1199
Shear Xm - Ym	=	0.4199	-0.4199
Shear Xm - Zm	=	0.7272	-0.7272
Torsion	=	0.0000	0.0000

```
Moment About Ym =      -39.2691         0.0000
Moment About Zm =       22.6727         0.0000
```

- Primary (Catenary) Cable Forces   Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	0.8542	10.8133	3	0.8462	0.0000	-0.1168
2	0.8463	10.7126				
3	0.8519	10.7838	7	-0.8462	-0.0012	-0.0983
4	0.8464	10.7141	7	0.0012	0.8407	-0.0984
5	0.8407	10.6423				
6	0.8487	10.7435	11	0.0000	-0.8407	-0.1168

- Secondary (Messenger) Cable Forces   Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.0000	6.6667	2	1.0000	0.0000	0.0000
2	1.0000	6.6667				
3	1.0000	6.6667	6	-1.0000	-0.0010	0.0000
4	1.0000	6.6667	6	0.0010	1.0000	0.0000
5	1.0000	6.6667				
6	1.0000	6.6667	10	0.0000	-1.0000	0.0000

- Light Member Forces

```
Member #   1
                          Node I         Node J

Axial Force      =       -0.1002         0.1002
Shear Xm - Ym    =        0.0000         0.0000
Shear Xm - Zm    =        0.0000         0.0000
Torsion          =        0.0000         0.0000
Moment About Ym =         0.0000         0.0000
Moment About Zm =         0.0000         0.0000

Member #   2
                          Node I         Node J

Axial Force      =       -0.1010         0.1010
Shear Xm - Ym    =        0.0000         0.0000
Shear Xm - Zm    =        0.0000         0.0000
Torsion          =        0.0000         0.0000
Moment About Ym =         0.0000         0.0000
Moment About Zm =         0.0000         0.0000

Member #   3
                          Node I         Node J

Axial Force      =       -0.1010         0.1010
Shear Xm - Ym    =        0.0000         0.0000
Shear Xm - Zm    =        0.0000         0.0000
Torsion          =        0.0000         0.0000
Moment About Ym =         0.0000         0.0000
Moment About Zm =         0.0000         0.0000

Member #   4
                          Node I         Node J

Axial Force      =       -0.1002         0.1002
Shear Xm - Ym    =        0.0000         0.0000
Shear Xm - Zm    =        0.0000         0.0000
Torsion          =        0.0000         0.0000
Moment About Ym =         0.0000         0.0000
Moment About Zm =         0.0000         0.0000
```

- Hanger (Connector) Member Forces

Member # 1

	Node I	Node J
Axial Force =	-0.1037	0.1037
Shear Xm - Ym =	0.0000	0.0000
Shear Xm - Zm =	0.0000	0.0000
Torsion =	0.0000	0.0000
Moment About Ym =	0.0000	0.0000
Moment About Zm =	0.0000	0.0000

Member # 2

	Node I	Node J
Axial Force =	-0.1043	0.1043
Shear Xm - Ym =	0.0000	0.0000
Shear Xm - Zm =	0.0000	0.0000
Torsion =	0.0000	0.0000
Moment About Ym =	0.0000	0.0000
Moment About Zm =	0.0000	0.0000

Member # 3

	Node I	Node J
Axial Force =	-0.1044	0.1044
Shear Xm - Ym =	0.0000	0.0000
Shear Xm - Zm =	0.0000	0.0000
Torsion =	0.0000	0.0000
Moment About Ym =	0.0000	0.0000
Moment About Zm =	0.0000	0.0000

Member # 4

	Node I	Node J
Axial Force =	-0.1038	0.1038
Shear Xm - Ym =	0.0000	0.0000
Shear Xm - Zm =	0.0000	0.0000
Torsion =	0.0000	0.0000
Moment About Ym =	0.0000	0.0000
Moment About Zm =	0.0000	0.0000

W I N D S O L U T I O N R E S U L T S

Final Coordinates

Node	X	Y	Z
1	600.0001	600.0001	0.0000
2	600.4519	600.0124	253.1200
3	600.6008	600.0163	307.1200
4	612.0001	600.0001	0.0000
5	1440.0000	600.0001	0.0000
6	1439.5484	600.3844	253.1200
7	1439.3996	600.5140	307.1200
8	1450.3920	606.0001	0.0000
9	1440.0000	1440.0000	0.0000
10	1439.9998	1439.6321	253.1200
11	1439.9997	1439.5055	307.1200
12	1450.3920	1446.0000	0.0000
13	889.4468	599.7816	266.6851
14	889.3973	610.4032	257.4098
15	889.2988	631.5573	238.9744
16	1077.8475	601.1175	264.9878

17	1077.8761	610.5627	257.5860
18	1077.9428	632.6614	240.2940
19	1439.6881	959.1429	264.5677
20	1439.6910	959.2467	252.5681
21	1439.6978	959.5564	224.5097
22	1439.8138	1152.5223	267.9377
23	1439.8106	1152.6403	253.7924
24	1439.8044	1152.9450	225.7340

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.4518	0.0123	0.0000	-0.0001	0.0027	0.0000
3	0.6007	0.0162	0.0000	-0.0001	0.0028	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.4516	0.3843	0.0000	-0.0024	-0.0027	0.0000
7	-0.6004	0.5139	0.0000	-0.0024	-0.0028	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
10	-0.0002	-0.3679	0.0000	0.0023	0.0000	0.0000
11	-0.0003	-0.4945	0.0000	0.0024	0.0000	0.0000
12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13	-0.0052	-0.2185	-0.5364	0.8528	0.0053	0.0000
14	-0.0547	10.4031	4.2898	0.8533	0.0053	0.0000
15	-0.1532	31.5572	13.9144	0.8543	0.0053	0.0000
16	-0.0400	1.1174	-0.1322	0.9060	-0.0039	0.0000
17	-0.0115	10.5626	4.4660	0.9063	-0.0039	0.0000
18	0.0553	32.6613	15.2340	0.9071	-0.0039	0.0000
19	-0.3119	-0.0783	-0.5523	0.0083	-0.0002	0.0000
20	-0.3090	0.0254	-0.5519	0.0094	-0.0002	0.0000
21	-0.3022	0.3352	-0.5503	0.0119	-0.0002	0.0000
22	-0.1862	-0.0966	0.6720	0.0079	0.0002	0.0000
23	-0.1894	0.0213	0.6724	0.0092	0.0002	0.0000
24	-0.1956	0.3260	0.6740	0.0117	0.0002	0.0000

- Frame Member Forces

Member # 1

		Node I	Node J
Axial Force	=	0.0385	-0.0385
Shear Xm - Ym	=	-2.3980	2.3980
Shear Xm - Zm	=	-0.0694	0.0694
Torsion	=	0.0000	0.0000
Moment About Ym	=	17.5481	0.0201
Moment About Zm	=	-631.6986	24.7122

Member # 2

		Node I	Node J
Axial Force	=	0.0673	-0.0673
Shear Xm - Ym	=	-0.4576	0.4576
Shear Xm - Zm	=	0.0004	-0.0004
Torsion	=	0.0000	0.0000
Moment About Ym	=	-0.0201	0.0000
Moment About Zm	=	-24.7122	0.0000

Member # 3

		Node I	Node J
Axial Force	=	0.1227	-0.1227
Shear Xm - Ym	=	1.1064	-1.1064
Shear Xm - Zm	=	-2.8780	2.8780
Torsion	=	0.0000	0.0000

```
Moment About Ym =     773.7300      -45.2556
Moment About Zm =     282.4273       -2.3841

Member #   4
                        Node I        Node J

Axial Force     =       0.1449       -0.1449
Shear Xm - Ym   =       0.0442       -0.0442
Shear Xm - Zm   =      -0.8381        0.8381
Torsion         =       0.0000        0.0000
Moment About Ym =      45.2556        0.0000
Moment About Zm =       2.3841        0.0000

Member #   5
                        Node I        Node J

Axial Force     =       0.1297       -0.1297
Shear Xm - Ym   =       0.8866       -0.8866
Shear Xm - Zm   =       1.5333       -1.5333
Torsion         =       0.0000        0.0000
Moment About Ym =    -432.0774       43.9666
Moment About Zm =     249.8434      -25.4229

Member #   6
                        Node I        Node J

Axial Force     =       0.1317       -0.1317
Shear Xm - Ym   =       0.4708       -0.4708
Shear Xm - Zm   =       0.8142       -0.8142
Torsion         =       0.0000        0.0000
Moment About Ym =     -43.9666        0.0000
Moment About Zm =      25.4229        0.0000
```

- Primary (Catenary) Cable Forces   Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	0.4623	5.8525	3	0.4579	-0.0004	-0.0641
2	0.4585	5.8036				
3	0.4611	5.8370	7	-0.4580	0.0008	-0.0534
4	0.7080	8.9625	7	0.0006	0.7031	-0.0834
5	0.8319	10.5299				
6	0.9494	12.0181	11	-0.0006	-0.9407	-0.1284

- Secondary (Messenger) Cable Forces Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.9422	12.9481	2	1.9407	0.0698	0.0288
2	1.9402	12.9345				
3	1.9415	12.9435	6	-1.9406	0.0546	0.0240
4	1.1811	7.8740	6	0.0005	1.1811	-0.0018
5	0.9962	6.6415				
6	0.8310	5.5400	10	-0.0005	-0.8310	0.0019

- Light Member Forces

```
Member #   1
Rotation Angle in X-Z Plane (Degrees) =   50.37
Rotation Angle in Y-Z Plane (Degrees) =    0.00

                        Node I        Node J

Axial Force     =      -0.0657        0.0657
Shear Xm - Ym   =      -0.0001        0.0001
Shear Xm - Zm   =      -0.0228        0.0228
Torsion         =       0.0000        0.0000
```

```
Moment About Ym =       0.6408         0.0000
Moment About Zm =      -0.0025         0.0000

Member #   2
Rotation Angle in X-Z Plane (Degrees) =   53.04
Rotation Angle in Y-Z Plane (Degrees) =    0.00

                        Node I         Node J

Axial Force     =      -0.0630         0.0630
Shear Xm - Ym   =       0.0001        -0.0001
Shear Xm - Zm   =      -0.0173         0.0173
Torsion         =       0.0000         0.0000
Moment About Ym =       0.4868         0.0000
Moment About Zm =       0.0017         0.0000

Member #   3
Rotation Angle in X-Z Plane (Degrees) =    1.70
Rotation Angle in Y-Z Plane (Degrees) =    0.00

                        Node I         Node J

Axial Force     =      -0.0984         0.0984
Shear Xm - Ym   =       0.0573        -0.0573
Shear Xm - Zm   =       0.0000         0.0000
Torsion         =       0.0000         0.0000
Moment About Ym =       0.0007         0.0000
Moment About Zm =       1.6089         0.0000

Member #   4
Rotation Angle in X-Z Plane (Degrees) =    1.70
Rotation Angle in Y-Z Plane (Degrees) =    0.00

                        Node I         Node J

Axial Force     =      -0.0976         0.0976
Shear Xm - Ym   =       0.0573        -0.0573
Shear Xm - Zm   =       0.0000         0.0000
Torsion         =       0.0000         0.0000
Moment About Ym =      -0.0006         0.0000
Moment About Zm =       1.6082         0.0000

- Hanger (Connector) Member Forces

Member #   1
                        Node I         Node J

Axial Force     =      -0.0582         0.0582
Shear Xm - Ym   =       0.0002        -0.0002
Shear Xm - Zm   =       0.0454        -0.0454
Torsion         =       0.0000         0.0000
Moment About Ym =       0.0000        -0.6408
Moment About Zm =       0.0000         0.0025

Member #   2
                        Node I         Node J

Axial Force     =      -0.0583         0.0583
Shear Xm - Ym   =      -0.0001         0.0001
Shear Xm - Zm   =       0.0406        -0.0406
Torsion         =       0.0000         0.0000
Moment About Ym =       0.0000        -0.4868
Moment About Zm =       0.0000        -0.0017

Member #   3
                        Node I         Node J

Axial Force     =      -0.1303         0.1303
Shear Xm - Ym   =      -0.1341         0.1341
```

```
Shear Xm - Zm     =        0.0001        -0.0001
Torsion           =        0.0000         0.0000
Moment About Ym =          0.0000        -0.0007
Moment About Zm =          0.0001        -1.6088

Member #    4
                           Node I         Node J

Axial Force       =       -0.1476         0.1476
Shear Xm - Ym     =       -0.1137         0.1137
Shear Xm - Zm     =        0.0000         0.0000
Torsion           =        0.0000         0.0000
Moment About Ym =          0.0000         0.0006
Moment About Zm =          0.0001        -1.6081
```

Example 6.3

The following is a three dimensional box structure as shown on Figure 6.14 below. The required clearance is 17 *ft*. The catenary cable has 1/4 in diameter and the messenger cable has 1/2 *in* diameter. The poles are made of prestress concrete and they are both of type *NVI*. The signal lights and sign arrangement as well as their type are also shown on Figure 6.14. The input as well as the output data files that are involved in the analysis of this particular system are also shown below.

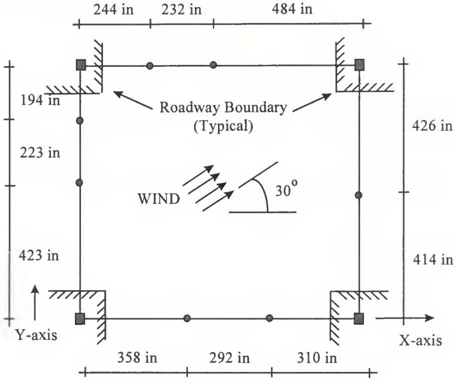

Figure 6. 14 Structure for Example 6.3

```
CONTROL
Title= Example 3 - Four Pole Configuration
NODES= 37
CLEAR= 265.12
CABLE=  8
SPEED=  40.0
ANGLE= 30
STATUS=3
:
CABLES
  1  3  7   S=    6.0   W=      0.23E-04   P=0
  2  2  6   T=    1.0   W=      0.43E-04   P=1
  3  7 11   S=    6.0   W=      0.23E-04   P=0
  4  6 10   T=    1.0   W=      0.43E-04   P=1
  5 11 15   S=    6.0   W=      0.23E-04   P=0
  6 10 14   T=    1.0   W=      0.43E-04   P=1
  7 15  3   S=    6.0   W=      0.23E-04   P=0
  8 14  2   T=    1.0   W=      0.43E-04   P=1
:
COORDINATE
    1 X= 0.7200000E+03 Y= 0.6000001E+03 Z= 0.0000000E+00
    2 X= 0.7200000E+03 Y= 0.6000001E+03 Z= 0.2531200E+03
    3 X= 0.7200000E+03 Y= 0.6000001E+03 Z= 0.3131200E+03
    4 X= 0.7316190E+03 Y= 0.6030001E+03 Z= 0.0000000E+00
    5 X= 0.1680000E+04 Y= 0.6000001E+03 Z= 0.0000000E+00
    6 X= 0.1680000E+04 Y= 0.6000001E+03 Z= 0.2531200E+03
    7 X= 0.1680000E+04 Y= 0.6000001E+03 Z= 0.3131200E+03
    8 X= 0.1691619E+04 Y= 0.6030001E+03 Z= 0.0000000E+00
    9 X= 0.1680000E+04 Y= 0.1440000E+04 Z= 0.0000000E+00
   10 X= 0.1680000E+04 Y= 0.1440000E+04 Z= 0.2531200E+03
   11 X= 0.1680000E+04 Y= 0.1440000E+04 Z= 0.3131200E+03
   12 X= 0.1691619E+04 Y= 0.1443000E+04 Z= 0.0000000E+00
   13 X= 0.7200000E+03 Y= 0.1440000E+04 Z= 0.0000000E+00
   14 X= 0.7200000E+03 Y= 0.1440000E+04 Z= 0.2531200E+03
   15 X= 0.7200000E+03 Y= 0.1440000E+04 Z= 0.3131200E+03
   16 X= 0.7316190E+03 Y= 0.1443000E+04 Z= 0.0000000E+00
   17 X= 0.1078025E+04 Y= 0.6000001E+03 C=    1
   18 X= 0.1078025E+04 Y= 0.6000001E+03 Z= 0.2531200E+03
   19 X= 0.1078025E+04 Y= 0.6000001E+03 Z= 0.2250600E+03
   20 X= 0.1370388E+04 Y= 0.6000001E+03 C=    1
   21 X= 0.1370388E+04 Y= 0.6000001E+03 Z= 0.2531200E+03
   22 X= 0.1370388E+04 Y= 0.6000001E+03 Z= 0.2250600E+03
   23 X= 0.1680000E+04 Y= 0.1013607E+04 C=    3
   24 X= 0.1680000E+04 Y= 0.1013607E+04 Z= 0.2531200E+03
   25 X= 0.1680000E+04 Y= 0.1013607E+04 Z= 0.2250600E+03
   26 X= 0.1196953E+04 Y= 0.1440000E+04 C=    5
   27 X= 0.1196953E+04 Y= 0.1440000E+04 Z= 0.2531200E+03
   28 X= 0.1196953E+04 Y= 0.1440000E+04 Z= 0.2250600E+03
   29 X= 0.9640535E+03 Y= 0.1440000E+04 C=    5
   30 X= 0.9640535E+03 Y= 0.1440000E+04 Z= 0.2531200E+03
   31 X= 0.9640535E+03 Y= 0.1440000E+04 Z= 0.2250600E+03
   32 X= 0.7200000E+03 Y= 0.1246505E+04 C=    7
   33 X= 0.7200000E+03 Y= 0.1246505E+04 Z= 0.2531200E+03
   34 X= 0.7200000E+03 Y= 0.1246505E+04 Z= 0.2250600E+03
   35 X= 0.7200000E+03 Y= 0.1023517E+04 C=    7
   36 X= 0.7200000E+03 Y= 0.1023517E+04 Z= 0.2531200E+03
   37 X= 0.7200000E+03 Y= 0.1023517E+04 Z= 0.2250600E+03
:
BOUNDARY
    1 DOF=f f f f f f
    2 DOF=r r r r r r
    3 DOF=r r r r r r
    4 DOF=f f f f f f
    5 DOF=f f f f f f
    6 DOF=r r r r r r
    7 DOF=r r r r r r
    8 DOF=f f f f f f
    9 DOF=f f f f f f
   10 DOF=r r r r r r
```

```
   11 DOF=r r r r r r
   12 DOF=f f f f f f
   13 DOF=f f f f f f
   14 DOF=r r r r r r
   15 DOF=r r r r r r
   16 DOF=f f f f f f
   17 DOF=r r r r r r
   18 DOF=r r r r r r
   19 DOF=r r r r r r
   20 DOF=r r r r r r
   21 DOF=r r r r r r
   22 DOF=r r r r r r
   23 DOF=r r r r r r
   24 DOF=r r r r r r
   25 DOF=r r r r r r
   26 DOF=r r r r r r
   27 DOF=r r r r r r
   28 DOF=r r r r r r
   29 DOF=r r r r r r
   30 DOF=r r r r r r
   31 DOF=r r r r r r
   32 DOF=r r r r r r
   33 DOF=r r r r r r
   34 DOF=r r r r r r
   35 DOF=r r r r r r
   36 DOF=r r r r r r
   37 DOF=r r r r r r
:
PRIMARY
   11,    1
  1 A=        0.0790 E=  0.27500E+05
   1     3,  17 M=  1 C=  1
   2    17,  20 M=  1 C=  1
   3    20,   7 M=  1 C=  1
   4     7,  23 M=  1 C=  3
   5    23,  11 M=  1 C=  3
   6    11,  26 M=  1 C=  5
   7    26,  29 M=  1 C=  5
   8    29,  15 M=  1 C=  5
   9    15,  32 M=  1 C=  7
  10    32,  35 M=  1 C=  7
  11    35,   3 M=  1 C=  7
:
SECONDARY
   11,    1
  1 A=        0.1500 E=  0.27500E+05
   1     2,  18 M=  1 C=  2
   2    18,  21 M=  1 C=  2
   3    21,   6 M=  1 C=  2
   4     6,  24 M=  1 C=  4
   5    24,  10 M=  1 C=  4
   6    10,  27 M=  1 C=  6
   7    27,  30 M=  1 C=  6
   8    30,  14 M=  1 C=  6
   9    14,  33 M=  1 C=  8
  10    33,  36 M=  1 C=  8
  11    36,   2 M=  1 C=  8
:
CONNECTORS
    7,    1
  1 A=        0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00
   1    17,  18,   1  M=  1
   2    20,  21,   1  M=  1
   3    23,  24,   5  M=  1
   4    26,  27,  13 M=  1
   5    29,  30,  13 M=  1
   6    32,  33,   1  M=  1
   7    35,  36,   1  M=  1
```

```
:
LIGHTS
    7,     2
  1 A=       0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00 S= 1 P=  0.48000E+03, 0.48000E+03
  2 A=       0.7990 E=  0.29000E+05 I=  0.31000E+00, 0.31000E+00  \
    J=  0.62000E+00 G=  0.00000E+00 S= 1 P=  0.48000E+03, 0.48000E+03
   1     18,  19,   1  M=  1
   2     21,  22,   1  M=  1
   3     24,  25,   5  M=  2
   4     27,  28,  13  M=  1
   5     30,  31,  13  M=  1
   6     33,  34,   1  M=  2
   7     36,  37,   1  M=  2
:
BEAM
    8,     1
  1  T= NVI    FC=      6000.0
   1      1,   2,   4 M=  1
   2      2,   3,   4 M=  1
   3      5,   6,   8 M=  1
   4      6,   7,   8 M=  1
   5      9,  10,  12 M=  1
   6     10,  11,  12 M=  1
   7     13,  14,  16 M=  1
   8     14,  15,  16 M=  1
:
SIGNS
   17 F=  0.00000E+00, 0.00000E+00,-0.11147E+00
   20 F=  0.00000E+00, 0.00000E+00,-0.11132E+00
   23 F=  0.00000E+00, 0.00000E+00,-0.11400E+00
   26 F=  0.00000E+00, 0.00000E+00,-0.11179E+00
   29 F=  0.00000E+00, 0.00000E+00,-0.11066E+00
   32 F=  0.00000E+00, 0.00000E+00,-0.11088E+00
   35 F=  0.00000E+00, 0.00000E+00,-0.11181E+00
:
WIND
:
LOADS
    3 F=  0.00000E+00, 0.00000E+00,-0.88122E-02
    7 F=  0.00000E+00, 0.00000E+00,-0.81563E-02
   11 F=  0.00000E+00, 0.00000E+00,-0.10249E-01
   15 F=  0.00000E+00, 0.00000E+00,-0.49427E-02
   17 F=  0.00000E+00, 0.00000E+00,-0.37192E-02
   18 F=  0.00000E+00, 0.00000E+00,-0.37192E-02
   19 F=  0.00000E+00, 0.00000E+00,-0.10403E+00
   20 F=  0.00000E+00, 0.00000E+00,-0.39183E-02
   21 F=  0.00000E+00, 0.00000E+00,-0.39183E-02
   22 F=  0.00000E+00, 0.00000E+00,-0.10348E+00
   23 F=  0.00000E+00, 0.00000E+00,-0.39157E-02
   24 F=  0.00000E+00, 0.00000E+00,-0.39157E-02
   25 F=  0.00000E+00, 0.00000E+00,-0.10617E+00
   26 F=  0.00000E+00, 0.00000E+00,-0.35101E-02
   27 F=  0.00000E+00, 0.00000E+00,-0.35101E-02
   28 F=  0.00000E+00, 0.00000E+00,-0.10477E+00
   29 F=  0.00000E+00, 0.00000E+00,-0.42929E-02
   30 F=  0.00000E+00, 0.00000E+00,-0.42929E-02
   31 F=  0.00000E+00, 0.00000E+00,-0.10208E+00
   32 F=  0.00000E+00, 0.00000E+00,-0.47395E-02
   33 F=  0.00000E+00, 0.00000E+00,-0.47395E-02
   34 F=  0.00000E+00, 0.00000E+00,-0.10140E+00
   35 F=  0.00000E+00, 0.00000E+00,-0.39152E-02
   36 F=  0.00000E+00, 0.00000E+00,-0.39152E-02
   37 F=  0.00000E+00, 0.00000E+00,-0.10398E+00
```

```
***********************************************************************
|                                                                     |
|      ##         #####      #              ##         ####           |
|     #  #          #        #             #  #       #               |
|    #    #         #        #            #    #       ####           |
|    ######         #        #            ######           #          |
|    #    #         #        #            #    #      #    #          |
|    #    #         #        ######       #    #       ####           |
|                                                                     |
|               Analysis of Traffic Lights And Signs                  |
|                                                                     |
|                           Version 4.0                               |
|                                                                     |
|          Developed by :                                             |
|                                                                     |
|          Dr Marc I. Hoit               Dr Ronald A. Cook            |
|          Mr Petros M. Christou         Ms Adeola K. Adediran        |
|                                                                     |
|                 Department of Civil Engineering                     |
|                      University of Florida                          |
|                      Gainesville, Fl 32608                          |
|                                                                     |
***********************************************************************
```

Input Data File = box
Output Data File = box.out

CONTROL DATA (More Information found in ATLAS User`s guide)

- Problem Title

  FOUR POLE CONFIGURATION

- Structural Parameters :

  Number of Nodes = 37
  Number of Cables = 8
  Roadway Clearance = 265.12

- Wind Data :

  Wind Speed (Miles per Hour) = 40.00
  Wind Direction (Angle from +ve X axis) = 30.0

- Nonlinear iteration Parameters :

  Number of Iterations (Shape Finder) = 500
  Number of Iterations (Gravity Solution) = 500
  Number of Iterations (Wind Solution) = 500
  Number of Loops for Shape Calculation = 5
  Number of Cycles (Shape-Stiffness Iteration) = 500
  Force Tolerance for Gravity Solution (%) = 5.00
  Force Tolerance for Wind Solution (%) = 3.00
  Pole Displacent Tolerance = 0.001000

ECHO OF NODAL POINT INPUT DATA

Nodal Point Coordinates				Boundary Conditions					
Node	X	Y	Z	Tx	Ty	Tz	Rx	Ry	Rz
1	720.000	600.000	0.000	F	F	F	F	F	F
2	720.000	600.000	253.120	R	R	R	R	R	R
3	720.000	600.000	313.120	R	R	R	R	R	R
4	731.619	603.000	0.000	F	F	F	F	F	F
5	1680.000	600.000	0.000	F	F	F	F	F	F

6	1680.000	600.000	253.120	R	R	R	R	R	R
7	1680.000	600.000	313.120	R	R	R	R	R	R
8	1691.619	603.000	0.000	F	F	F	F	F	F
9	1680.000	1440.000	0.000	F	F	F	F	F	F
10	1680.000	1440.000	253.120	R	R	R	R	R	R
11	1680.000	1440.000	313.120	R	R	R	R	R	R
12	1691.619	1443.000	0.000	F	F	F	F	F	F
13	720.000	1440.000	0.000	F	F	F	F	F	F
14	720.000	1440.000	253.120	R	R	R	R	R	R
15	720.000	1440.000	313.120	R	R	R	R	R	R
16	731.619	1443.000	0.000	F	F	F	F	F	F
17	1078.025	600.000	259.239	R	R	R	R	R	R
18	1078.025	600.000	253.120	R	R	R	R	R	R
19	1078.025	600.000	225.060	R	R	R	R	R	R
20	1370.388	600.000	262.778	R	R	R	R	R	R
21	1370.388	600.000	253.120	R	R	R	R	R	R
22	1370.388	600.000	225.060	R	R	R	R	R	R
23	1680.000	1013.607	262.732	R	R	R	R	R	R
24	1680.000	1013.607	253.120	R	R	R	R	R	R
25	1680.000	1013.607	225.060	R	R	R	R	R	R
26	1196.953	1440.000	255.522	R	R	R	R	R	R
27	1196.953	1440.000	253.120	R	R	R	R	R	R
28	1196.953	1440.000	225.060	R	R	R	R	R	R
29	964.053	1440.000	269.438	R	R	R	R	R	R
30	964.053	1440.000	253.120	R	R	R	R	R	R
31	964.053	1440.000	225.060	R	R	R	R	R	R
32	720.000	1246.505	277.378	R	R	R	R	R	R
33	720.000	1246.505	253.120	R	R	R	R	R	R
34	720.000	1246.505	225.060	R	R	R	R	R	R
35	720.000	1023.517	262.724	R	R	R	R	R	R
36	720.000	1023.517	253.120	R	R	R	R	R	R
37	720.000	1023.517	225.060	R	R	R	R	R	R

ECHO OF ELEMENT INPUT DATA

1. Pole/Beam Element Data

Number of Property Lines = 1

Property Line = 1
Pole type = NVI
Concrete Strength, F`c (psi) = 6000.00

NOTE : The properties used in the analysis were obtained at the effective heights of the poles and are provided below. For more information refer to the report that accompanies the program.

Pole/Beam Connectivity and Properties Used

	Nodes				Properties					
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	1	2	4	1	155.47	4415.20	4232.69	4232.69	8465.38	1698.15
2	2	3	4	1	124.03	4415.20	2022.13	2022.13	4044.27	1698.15
3	5	6	8	1	155.47	4415.20	4232.69	4232.69	8465.38	1698.15
4	6	7	8	1	124.03	4415.20	2022.13	2022.13	4044.27	1698.15
5	9	10	12	1	155.47	4415.20	4232.69	4232.69	8465.38	1698.15
6	10	11	12	1	124.03	4415.20	2022.13	2022.13	4044.27	1698.15
7	13	14	16	1	155.47	4415.20	4232.69	4232.69	8465.38	1698.15
8	14	15	16	1	124.03	4415.20	2022.13	2022.13	4044.27	1698.15

2. Primary Cable Element Data

Number of Property Lines = 1

Primary Cable Connectivity and Properties

Mem	Nodes I	J	Mat	Cable	Properties Area	E
1	3	17	1	1	0.0790	27500.0
2	17	20	1	1	0.0790	27500.0
3	20	7	1	1	0.0790	27500.0
4	7	23	1	3	0.0790	27500.0
5	23	11	1	3	0.0790	27500.0
6	11	26	1	5	0.0790	27500.0
7	26	29	1	5	0.0790	27500.0
8	29	15	1	5	0.0790	27500.0
9	15	32	1	7	0.0790	27500.0
10	32	35	1	7	0.0790	27500.0
11	35	3	1	7	0.0790	27500.0

3. Secondary Cable Element Data

Number of Property Lines = 1

Secondary Cable Connectivity and Properties

Mem	Nodes I	J	Mat	Cable	Properties Area	E
1	2	18	1	2	0.1500	27500.0
2	18	21	1	2	0.1500	27500.0
3	21	6	1	2	0.1500	27500.0
4	6	24	1	4	0.1500	27500.0
5	24	10	1	4	0.1500	27500.0
6	10	27	1	6	0.1500	27500.0
7	27	30	1	6	0.1500	27500.0
8	30	14	1	6	0.1500	27500.0
9	14	33	1	8	0.1500	27500.0
10	33	36	1	8	0.1500	27500.0
11	36	2	1	8	0.1500	27500.0

4. Connector Element Data

Number of Property Lines = 1

Connector Connectivity and Properties

Mem	Nodes I	J	K	Mat	Area	E	Properties I33	I22	J	G
1	17	18	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
2	20	21	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	23	24	5	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
4	26	27	13	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
5	29	30	13	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
6	32	33	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
7	35	36	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8

5. Light Element Data

Number of Property Lines = 2

Property Line = 1
Projected area on X-Z plane = 480.00
Projected area on Y-Z plane = 480.00

Property Line = 2
Projected area on X-Z plane = 480.00
Projected area on Y-Z plane = 480.00

Light Connectivity and Properties

	Nodes				Properties					
Mem	I	J	K	Mat	Area	E	I33	I22	J	G
1	18	19	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
2	21	22	1	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
3	24	25	5	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
4	27	28	13	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
5	30	31	13	1	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
6	33	34	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8
7	36	37	1	2	0.7990	29000.0	0.3100	0.3100	0.6200	11153.8

CONCENTRATED APPLIED LOADS

- Sign weights

Node	X	Y	Z
3	0.00000	0.00000	-0.00881
7	0.00000	0.00000	-0.00816
11	0.00000	0.00000	-0.01025
15	0.00000	0.00000	-0.00494
17	0.00000	0.00000	-0.00372
18	0.00000	0.00000	-0.00372
19	0.00000	0.00000	-0.10403
20	0.00000	0.00000	-0.00392
21	0.00000	0.00000	-0.00392
22	0.00000	0.00000	-0.10348
23	0.00000	0.00000	-0.00392
24	0.00000	0.00000	-0.00392
25	0.00000	0.00000	-0.10617
26	0.00000	0.00000	-0.00351
27	0.00000	0.00000	-0.00351
28	0.00000	0.00000	-0.10477
29	0.00000	0.00000	-0.00429
30	0.00000	0.00000	-0.00429
31	0.00000	0.00000	-0.10208
32	0.00000	0.00000	-0.00474
33	0.00000	0.00000	-0.00474
34	0.00000	0.00000	-0.10140
35	0.00000	0.00000	-0.00392
36	0.00000	0.00000	-0.00392
37	0.00000	0.00000	-0.10398

G R A V I T Y   S O L U T I O N   R E S U L T S

Final Coordinates

Node	X	Y	Z
1	720.0000	600.0001	0.0000
2	720.5972	600.5678	253.1199
3	720.8254	600.7840	313.1199
4	731.6190	603.0001	0.0000
5	1680.0000	600.0001	0.0000
6	1679.4027	600.4835	253.1199
7	1679.1745	600.6654	313.1199
8	1691.6190	603.0001	0.0000
9	1680.0000	1440.0000	0.0000
10	1679.3820	1439.5162	253.1199
11	1679.1452	1439.3342	313.1199
12	1691.6190	1443.0000	0.0000
13	720.0000	1440.0000	0.0000
14	720.6164	1439.4334	253.1199
15	720.8525	1439.2176	313.1199
16	731.6190	1443.0000	0.0000

17	1078.0244	600.0001	265.1229
18	1078.0244	600.0001	253.1229
19	1078.0244	600.0001	225.0627
20	1370.6647	600.0001	267.2378
21	1370.6647	600.0001	253.1227
22	1370.6646	600.0001	225.0626
23	1680.0000	1013.5522	265.1214
24	1680.0000	1013.5522	253.1213
25	1680.0000	1013.5522	225.0612
26	1196.4372	1440.0000	265.1231
27	1196.4372	1440.0000	253.1230
28	1196.4372	1440.0000	225.0629
29	963.2612	1440.0000	272.9557
30	963.2612	1440.0000	253.1216
31	963.2612	1440.0000	225.0614
32	720.0000	1247.5084	274.8387
33	720.0000	1247.5084	253.1211
34	720.0000	1247.5084	225.0609
35	720.0000	1024.1620	265.1211
36	720.0000	1024.1620	253.1210
37	720.0000	1024.1620	225.0609

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.5972	0.5677	-0.0001	-0.0035	0.0037	0.0000
3	0.8254	0.7839	-0.0001	-0.0037	0.0039	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.5973	0.4834	-0.0001	-0.0030	-0.0037	0.0000
7	-0.8255	0.6653	-0.0001	-0.0031	-0.0039	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
10	-0.6180	-0.4838	-0.0001	0.0030	-0.0038	0.0000
11	-0.8548	-0.6658	-0.0001	0.0031	-0.0040	0.0000
12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
14	0.6164	-0.5666	-0.0001	0.0035	0.0038	0.0000
15	0.8525	-0.7824	-0.0001	0.0036	0.0040	0.0000
16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
17	0.0000	0.0000	0.0029	0.0000	0.0000	0.0000
18	0.0000	0.0000	0.0029	0.0000	0.0000	0.0000
19	0.0000	0.0000	0.0027	0.0000	0.0000	0.0000
20	0.0000	0.0000	0.0028	0.0000	0.0000	0.0000
21	0.0000	0.0000	0.0027	0.0000	0.0000	0.0000
22	0.0000	0.0000	0.0026	0.0000	0.0000	0.0000
23	0.0000	0.0000	0.0014	0.0000	0.0000	0.0000
24	0.0000	0.0000	0.0013	0.0000	0.0000	0.0000
25	0.0000	0.0000	0.0012	0.0000	0.0000	0.0000
26	0.0000	0.0000	0.0031	0.0000	0.0000	0.0000
27	0.0000	0.0000	0.0030	0.0000	0.0000	0.0000
28	0.0000	0.0000	0.0029	0.0000	0.0000	0.0000
29	0.0000	0.0000	0.0017	0.0000	0.0000	0.0000
30	0.0000	0.0000	0.0016	0.0000	0.0000	0.0000
31	0.0000	0.0000	0.0014	0.0000	0.0000	0.0000
32	0.0000	0.0000	0.0012	0.0000	0.0000	0.0000
33	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000
34	0.0000	0.0000	0.0009	0.0000	0.0000	0.0000
35	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000
36	0.0000	0.0000	0.0010	0.0000	0.0000	0.0000
37	0.0000	0.0000	0.0009	0.0000	0.0000	0.0000

- Frame Member Forces

Member # 1

	Node I	Node J
Axial Force =	0.1953	-0.1953
Shear Xm - Ym =	-2.1558	2.1558
Shear Xm - Zm =	-1.2090	1.2090
Torsion =	0.0000	0.0000
Moment About Ym =	335.5439	-29.5317
Moment About Zm =	-601.9979	56.3196

Member # 2

	Node I	Node J
Axial Force =	0.1954	-0.1954
Shear Xm - Ym =	-0.9387	0.9387
Shear Xm - Zm =	-0.4922	0.4922
Torsion =	0.0000	0.0000
Moment About Ym =	29.5317	0.0000
Moment About Zm =	-56.3196	0.0000

Member # 3

	Node I	Node J
Axial Force =	0.1830	-0.1830
Shear Xm - Ym =	1.3550	-1.3550
Shear Xm - Zm =	-1.8935	1.8935
Torsion =	0.0000	0.0000
Moment About Ym =	519.9059	-40.6136
Moment About Zm =	381.1861	-38.2132

Member # 4

	Node I	Node J
Axial Force =	0.1831	-0.1831
Shear Xm - Ym =	0.6369	-0.6369
Shear Xm - Zm =	-0.6769	0.6769
Torsion =	0.0000	0.0000
Moment About Ym =	40.6136	0.0000
Moment About Zm =	38.2132	0.0000

Member # 5

	Node I	Node J
Axial Force =	0.1499	-0.1499
Shear Xm - Ym =	2.1537	-2.1537
Shear Xm - Zm =	0.9888	-0.9888
Torsion =	0.0000	0.0000
Moment About Ym =	-266.5418	16.2690
Moment About Zm =	601.3670	-56.2111

Member # 6

	Node I	Node J
Axial Force =	0.1499	-0.1499
Shear Xm - Ym =	0.9369	-0.9369
Shear Xm - Zm =	0.2711	-0.2711
Torsion =	0.0000	0.0000
Moment About Ym =	-16.2690	0.0000
Moment About Zm =	56.2111	0.0000

Member # 7

	Node I	Node J
Axial Force =	0.2858	-0.2858
Shear Xm - Ym =	-1.3486	1.3486
Shear Xm - Zm =	2.1110	-2.1110
Torsion =	0.0000	0.0000

Moment About Ym	=	-588.0602	53.7243
Moment About Zm	=	-379.3066	37.9521

Member # 8

		Node I	Node J
Axial Force	=	0.2861	-0.2861
Shear Xm - Ym	=	-0.6325	0.6325
Shear Xm - Zm	=	0.8954	-0.8954
Torsion	=	0.0000	0.0000
Moment About Ym	=	-53.7243	0.0000
Moment About Zm	=	-37.9521	0.0000

- Primary (Catenary) Cable Forces   Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	0.7960	10.0756	3	0.7889	-0.0017	-0.1060
2	0.7889	9.9865				
3	0.7975	10.0955	7	-0.7889	-0.0017	-0.1173
4	0.5020	6.3543	7	0.0010	0.4986	-0.0580
5	0.5018	6.3518	11	0.0010	-0.4986	-0.0562
6	0.8456	10.7032	11	-0.8414	0.0012	-0.0837
7	0.8419	10.6568				
8	0.8528	10.7950	15	0.8413	0.0027	-0.1394
9	0.7276	9.2105	15	-0.0032	-0.7135	-0.1425
10	0.7143	9.0421				
11	0.7182	9.0913	3	-0.0014	0.7136	-0.0809

- Secondary (Messenger) Cable Forces Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.0000	6.6667	2	1.0000	-0.0016	0.0000
2	1.0000	6.6667				
3	1.0000	6.6667	6	-1.0000	-0.0016	0.0000
4	1.0000	6.6667	6	0.0014	1.0000	0.0000
5	1.0000	6.6667	10	0.0015	-1.0000	0.0000
6	1.0000	6.6667	10	-1.0000	0.0010	0.0000
7	1.0000	6.6667				
8	1.0000	6.6667	14	1.0000	0.0023	0.0000
9	1.0000	6.6667	14	-0.0032	-1.0000	0.0000
10	1.0000	6.6667				
11	1.0000	6.6667	2	-0.0014	1.0000	0.0000

- Light Member Forces

Member # 1

		Node I	Node J
Axial Force	=	-0.1040	0.1040
Shear Xm - Ym	=	0.0000	0.0000
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	0.0000	0.0000

Member # 2

		Node I	Node J
Axial Force	=	-0.1035	0.1035
Shear Xm - Ym	=	0.0000	0.0000
Shear Xm - Zm	=	0.0000	0.0000
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0000	0.0000
Moment About Zm	=	0.0000	0.0000

```
Member #   3
                          Node I          Node J

Axial Force      =       -0.1062          0.1062
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

Member #   4
                          Node I          Node J

Axial Force      =       -0.1048          0.1048
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

Member #   5
                          Node I          Node J

Axial Force      =       -0.1021          0.1021
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

Member #   6
                          Node I          Node J

Axial Force      =       -0.1014          0.1014
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

Member #   7
                          Node I          Node J

Axial Force      =       -0.1040          0.1040
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

- Hanger (Connector) Member Forces

Member #   1
                          Node I          Node J

Axial Force      =       -0.1078          0.1078
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
Moment About Ym =         0.0000          0.0000
Moment About Zm =         0.0000          0.0000

Member #   2
                          Node I          Node J

Axial Force      =       -0.1074          0.1074
Shear Xm - Ym    =        0.0000          0.0000
Shear Xm - Zm    =        0.0000          0.0000
Torsion          =        0.0000          0.0000
```

```
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000

Member #   3
                        Node I        Node J

Axial Force     =      -0.1101        0.1101
Shear Xm - Ym   =       0.0000        0.0000
Shear Xm - Zm   =       0.0000        0.0000
Torsion         =       0.0000        0.0000
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000

Member #   4
                        Node I        Node J

Axial Force     =      -0.1083        0.1083
Shear Xm - Ym   =       0.0000        0.0000
Shear Xm - Zm   =       0.0000        0.0000
Torsion         =       0.0000        0.0000
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000

Member #   5
                        Node I        Node J

Axial Force     =      -0.1065        0.1065
Shear Xm - Ym   =       0.0000        0.0000
Shear Xm - Zm   =       0.0000        0.0000
Torsion         =       0.0000        0.0000
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000

Member #   6
                        Node I        Node J

Axial Force     =      -0.1063        0.1063
Shear Xm - Ym   =       0.0000        0.0000
Shear Xm - Zm   =       0.0000        0.0000
Torsion         =       0.0000        0.0000
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000

Member #   7
                        Node I        Node J

Axial Force     =      -0.1080        0.1080
Shear Xm - Ym   =       0.0000        0.0000
Shear Xm - Zm   =       0.0000        0.0000
Torsion         =       0.0000        0.0000
Moment About Ym =       0.0000        0.0000
Moment About Zm =       0.0000        0.0000
```

W I N D   S O L U T I O N   R E S U L T S

Final Coordinates

Node	X	Y	Z
1	720.0000	600.0001	0.0000
2	720.6082	600.6001	253.1199
3	720.8390	600.8268	313.1199
4	731.6190	603.0001	0.0000
5	1680.0000	600.0001	0.0000

6	1679.4057	600.4977	253.1199
7	1679.1776	600.6841	313.1199
8	1691.6190	603.0001	0.0000
9	1680.0000	1440.0000	0.0000
10	1679.3859	1439.5079	253.1199
11	1679.1498	1439.3232	313.1199
12	1691.6190	1443.0000	0.0000
13	720.0000	1440.0000	0.0000
14	720.6329	1439.4087	253.1199
15	720.8739	1439.1846	313.1199
16	731.6190	1443.0000	0.0000
17	1077.9656	601.8624	264.6878
18	1078.0393	602.6865	252.7163
19	1078.2442	604.6158	224.7233
20	1370.5966	601.7384	267.6872
21	1370.6799	602.6566	253.6022
22	1370.8798	604.4848	225.6024
23	1681.2410	1013.5536	265.0649
24	1683.4601	1013.5565	253.2718
25	1688.6562	1013.5739	225.6970
26	1196.4112	1440.6512	265.3447
27	1196.4533	1441.5120	253.3756
28	1196.5839	1443.5265	225.3882
29	963.2200	1440.1550	272.6221
30	963.2765	1441.3032	252.8213
31	963.3949	1442.9320	224.8087
32	722.1586	1247.4928	274.7762
33	725.3723	1247.5355	253.2977
34	729.5409	1247.6041	225.5491
35	723.4735	1024.1610	264.9032
36	725.8533	1024.1817	253.1415
37	731.4238	1024.2410	225.6399

Final Displacements

Node	Tx	Ty	Tz	Rot-X	Rot-Y	Rot-Z
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.6082	0.6000	-0.0001	-0.0037	0.0038	0.0000
3	0.8390	0.8267	-0.0001	-0.0038	0.0039	0.0000
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
6	-0.5943	0.4976	-0.0001	-0.0030	-0.0037	0.0000
7	-0.8224	0.6840	-0.0001	-0.0031	-0.0039	0.0000
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
10	-0.6141	-0.4921	-0.0001	0.0030	-0.0038	0.0000
11	-0.8502	-0.6768	-0.0001	0.0031	-0.0040	0.0000
12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
14	0.6329	-0.5913	-0.0001	0.0036	0.0039	0.0000
15	0.8739	-0.8154	-0.0001	0.0038	0.0041	0.0000
16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
17	-0.0588	1.8623	-0.4322	0.0687	-0.0060	0.0000
18	0.0149	2.6864	-0.4037	0.0688	-0.0065	0.0000
19	0.2198	4.6157	-0.3367	0.0688	-0.0077	-0.0001
20	-0.0681	1.7383	0.4522	0.0651	-0.0057	0.0000
21	0.0153	2.6565	0.4822	0.0651	-0.0063	0.0000
22	0.2151	4.4847	0.5424	0.0652	-0.0075	-0.0001
23	1.2410	0.0014	-0.0551	0.0002	-0.1859	0.0000
24	3.4601	0.0043	0.1518	0.0004	-0.1861	0.0000
25	8.6562	0.0217	0.6370	0.0007	-0.1863	0.0001
26	-0.0259	0.6512	0.2247	0.0718	-0.0033	0.0000
27	0.0162	1.5120	0.2556	0.0718	-0.0039	0.0000
28	0.1468	3.5265	0.3282	0.0719	-0.0051	-0.0001
29	-0.0412	0.1550	-0.3320	0.0579	-0.0026	0.0000
30	0.0153	1.3032	-0.2987	0.0580	-0.0034	-0.0001
31	0.1337	2.9320	-0.2513	0.0581	-0.0046	-0.0001

32	2.1586	-0.0155	-0.0613	0.0019	-0.1484	0.0000
33	5.3723	0.0272	0.1777	0.0022	-0.1488	0.0000
34	9.5409	0.0957	0.4891	0.0026	-0.1493	0.0001
35	3.4735	-0.0011	-0.2168	0.0017	-0.1996	0.0000
36	5.8533	0.0197	0.0215	0.0019	-0.1997	0.0000
37	11.4238	0.0790	0.5799	0.0023	-0.1999	0.0001

- Frame Member Forces

Member # 1

	Node I	Node J
Axial Force =	0.1821	-0.1821
Shear Xm - Ym =	-2.2474	2.2474
Shear Xm - Zm =	-1.3199	1.3199
Torsion =	0.0000	0.0000
Moment About Ym =	361.5880	-27.4837
Moment About Zm =	-620.6595	51.7850

Member # 2

	Node I	Node J
Axial Force =	0.1809	-0.1809
Shear Xm - Ym =	-0.8631	0.8631
Shear Xm - Zm =	-0.4581	0.4581
Torsion =	0.0000	0.0000
Moment About Ym =	27.4837	0.0000
Moment About Zm =	-51.7850	0.0000

Member # 3

	Node I	Node J
Axial Force =	0.1825	-0.1825
Shear Xm - Ym =	1.3160	-1.3160
Shear Xm - Zm =	-1.9448	1.9448
Torsion =	0.0000	0.0000
Moment About Ym =	531.8518	-39.5734
Moment About Zm =	374.1461	-41.0439

Member # 4

	Node I	Node J
Axial Force =	0.1844	-0.1844
Shear Xm - Ym =	0.6841	-0.6841
Shear Xm - Zm =	-0.6596	0.6596
Torsion =	0.0000	0.0000
Moment About Ym =	39.5734	0.0000
Moment About Zm =	41.0439	0.0000

Member # 5

	Node I	Node J
Axial Force =	0.1516	-0.1516
Shear Xm - Ym =	2.1345	-2.1345
Shear Xm - Zm =	1.0284	-1.0284
Torsion =	0.0000	0.0000
Moment About Ym =	-275.1633	14.8587
Moment About Zm =	598.7062	-58.4158

Member # 6

	Node I	Node J
Axial Force =	0.1525	-0.1525
Shear Xm - Ym =	0.9736	-0.9736
Shear Xm - Zm =	0.2476	-0.2476
Torsion =	0.0000	0.0000
Moment About Ym =	-14.8587	0.0000
Moment About Zm =	58.4158	0.0000

Member # 7

	Node I	Node J
Axial Force =	0.2739	-0.2739
Shear Xm - Ym =	-1.3969	1.3969
Shear Xm - Zm =	2.2204	-2.2204
Torsion =	0.0000	0.0000
Moment About Ym =	-613.6396	51.6243
Moment About Zm =	-389.1077	35.5162

Member # 8

	Node I	Node J
Axial Force =	0.2736	-0.2736
Shear Xm - Ym =	-0.5919	0.5919
Shear Xm - Zm =	0.8604	-0.8604
Torsion =	0.0000	0.0000
Moment About Ym =	-51.6243	0.0000
Moment About Zm =	-35.5162	0.0000

- Primary (Catenary) Cable Forces Primary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	0.7239	9.1636	3	0.7174	0.0021	-0.0973
2	0.7782	9.8507				
3	0.8388	10.6181	7	-0.8299	0.0028	-0.1222
4	0.4681	5.9252	7	0.0023	0.4649	-0.0541
5	0.4889	6.1882	11	0.0024	-0.4858	-0.0548
6	0.8876	11.2360	11	-0.8833	0.0024	-0.0874
7	0.8220	10.4045				
8	0.7950	10.0633	15	0.7841	0.0031	-0.1310
9	0.7025	8.8926	15	0.0046	-0.6889	-0.1378
10	0.6784	8.5875				
11	0.6617	8.3758	3	0.0041	0.6574	-0.0749

- Secondary (Messenger) Cable Forces Secondary Cable Reactions on Poles

Member	Force	Stress	Node	Fx	Fy	Fz
1	1.1108	7.4054	2	1.1108	0.0065	-0.0013
2	1.0230	6.8199				
3	0.9443	6.2953	6	-0.9443	0.0066	0.0015
4	1.0802	7.2014	6	0.0106	1.0801	0.0004
5	1.0504	7.0025	10	0.0100	-1.0503	0.0004
6	0.9392	6.2613	10	-0.9392	0.0039	0.0005
7	1.0279	6.8529				
8	1.0924	7.2827	14	1.0924	0.0085	-0.0013
9	1.1253	7.5022	14	0.0278	-1.1250	0.0010
10	1.1451	7.6341				
11	1.1745	7.8301	2	0.0145	1.1744	0.0001

- Light Member Forces

Member # 1
Rotation Angle in X-Z Plane (Degrees) = 5.92
Rotation Angle in Y-Z Plane (Degrees) = 2.02

	Node I	Node J
Axial Force =	-0.1024	0.1024
Shear Xm - Ym =	0.0277	-0.0277
Shear Xm - Zm =	-0.0020	0.0020
Torsion =	0.0000	0.0000
Moment About Ym =	0.0549	0.0000
Moment About Zm =	0.7785	0.0000

Member # 2
Rotation Angle in X-Z Plane (Degrees) = 4.81
Rotation Angle in Y-Z Plane (Degrees) = 2.02

		Node I	Node J
Axial Force	=	-0.1020	0.1020
Shear Xm - Ym	=	0.0277	-0.0277
Shear Xm - Zm	=	-0.0024	0.0024
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0660	0.0000
Moment About Zm	=	0.7779	0.0000

Member # 3
Rotation Angle in X-Z Plane (Degrees) = 1.65
Rotation Angle in Y-Z Plane (Degrees) = 12.23

		Node I	Node J
Axial Force	=	-0.1033	0.1033
Shear Xm - Ym	=	0.0091	-0.0091
Shear Xm - Zm	=	0.0063	-0.0063
Torsion	=	0.0000	0.0000
Moment About Ym	=	-0.1778	0.0000
Moment About Zm	=	0.2544	0.0000

Member # 4
Rotation Angle in X-Z Plane (Degrees) = 5.18
Rotation Angle in Y-Z Plane (Degrees) = 1.88

		Node I	Node J
Axial Force	=	-0.1034	0.1034
Shear Xm - Ym	=	0.0275	-0.0275
Shear Xm - Zm	=	-0.0016	0.0016
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0442	0.0000
Moment About Zm	=	0.7710	0.0000

Member # 5
Rotation Angle in X-Z Plane (Degrees) = 4.33
Rotation Angle in Y-Z Plane (Degrees) = 1.78

		Node I	Node J
Axial Force	=	-0.1008	0.1008
Shear Xm - Ym	=	0.0274	-0.0274
Shear Xm - Zm	=	-0.0031	0.0031
Torsion	=	0.0000	0.0000
Moment About Ym	=	0.0881	0.0000
Moment About Zm	=	0.7695	0.0000

Member # 6
Rotation Angle in X-Z Plane (Degrees) = 1.65
Rotation Angle in Y-Z Plane (Degrees) = 10.00

		Node I	Node J
Axial Force	=	-0.0993	0.0993
Shear Xm - Ym	=	0.0092	-0.0092
Shear Xm - Zm	=	0.0113	-0.0113
Torsion	=	0.0000	0.0000
Moment About Ym	=	-0.3177	0.0000
Moment About Zm	=	0.2593	0.0000

Member # 7
Rotation Angle in X-Z Plane (Degrees) = 1.65
Rotation Angle in Y-Z Plane (Degrees) = 13.22

	Node I	Node J
Axial Force =	-0.1008	0.1008
Shear Xm - Ym =	0.0092	-0.0092
Shear Xm - Zm =	0.0052	-0.0052
Torsion =	0.0000	0.0000
Moment About Ym =	-0.1456	0.0000
Moment About Zm =	0.2584	0.0000

- Hanger (Connector) Member Forces

Member # 1

	Node I	Node J
Axial Force =	-0.1210	0.1210
Shear Xm - Ym =	-0.0649	0.0649
Shear Xm - Zm =	0.0046	-0.0046
Torsion =	-0.0001	0.0001
Moment About Ym =	0.0000	-0.0550
Moment About Zm =	0.0000	-0.7785

Member # 2

	Node I	Node J
Axial Force =	-0.1322	0.1322
Shear Xm - Ym =	-0.0551	0.0551
Shear Xm - Zm =	0.0047	-0.0047
Torsion =	-0.0002	0.0002
Moment About Ym =	0.0000	-0.0661
Moment About Zm =	0.0000	-0.7779

Member # 3

	Node I	Node J
Axial Force =	-0.1233	0.1233
Shear Xm - Ym =	-0.0212	0.0212
Shear Xm - Zm =	-0.0148	0.0148
Torsion =	0.0001	-0.0001
Moment About Ym =	0.0000	0.1778
Moment About Zm =	0.0000	-0.2544

Member # 4

	Node I	Node J
Axial Force =	-0.1291	0.1291
Shear Xm - Ym =	-0.0642	0.0642
Shear Xm - Zm =	0.0037	-0.0037
Torsion =	-0.0001	0.0001
Moment About Ym =	0.0000	-0.0443
Moment About Zm =	0.0000	-0.7710

Member # 5

	Node I	Node J
Axial Force =	-0.1269	0.1269
Shear Xm - Ym =	-0.0388	0.0388
Shear Xm - Zm =	0.0044	-0.0044
Torsion =	-0.0002	0.0002
Moment About Ym =	0.0000	-0.0882
Moment About Zm =	0.0000	-0.7695

Member # 6

	Node I	Node J
Axial Force =	-0.1282	0.1282
Shear Xm - Ym =	-0.0119	0.0119
Shear Xm - Zm =	-0.0146	0.0146
Torsion =	0.0003	-0.0003
Moment About Ym =	0.0000	0.3178

Moment About Zm =	0.0001	-0.2593

Member # 7

	Node I	Node J
Axial Force =	-0.1193	0.1193
Shear Xm - Ym =	-0.0215	0.0215
Shear Xm - Zm =	-0.0121	0.0121
Torsion =	0.0001	-0.0001
Moment About Ym =	0.0000	0.1456
Moment About Zm =	0.0000	-0.2584

CHAPTER 7
CONCLUSIONS

7.1 Summary

This dissertation presents a solution strategy for the analysis of Frame-Cable (FC) structures. The solution strategy is an iterative technique and consists of two methods, the Force Density Method (FDM) for the analysis of cable networks and the Direct Stiffness Method (DSM) which are executed successively to obtain an approximate solution for the structure. When the approximate solution is obtained, the Non-Linear Direct Stiffness Method (NLDSM) is implemented to obtain the final solution of the structure starting from the approximate solution obtained by the iterative process (FDM -DSM). The NLDSM is implemented to allow for large displacements. The solution strategy that is presented herein is heavily computational and therefore the use of the digital computer is necessary.

Cable elements cannot support any compressive forces. Rather they become slack. This dissertation also presents the implementation of a slack cable in the FDM. The implementation of this particular element is necessary to completely model the true behavior of the cable elements which may become slack depending on the nature of the applied loads.

The analysis of Frame-Cable structures is not an easy task. The analysis becomes even more complicated when the individual structures under consideration have their own peculiarities i.e. the element connectivity, the nature of the applied load etc. It is therefore very difficult to develop an analysis technique which can directly be applied on every kind of Frame-Cable structures without any modification. The solution strategy that is

presented in this work is no exception to the “general” rule. Therefore, it needs to be adjusted to the peculiarities of the individual system before it is implemented for the analysis of any Frame-Cable structure.

The computer program ATLAS (Analysis of Traffic signal Lights And Signs' supports) was developed to perform the analysis of the signal lights and signs that are supported by the dual cable system. ATLAS performs the analysis (and design) of the aforementioned systems considering the gravity load on the system as well as the applied wind load. The wind load on the structure is based on the maximum wind that the structure is expected to experience during its intended life. The wind load is not a function of time; rather it is a function of the area and the rotation angle of the element that is applied on. ATLAS, is not general. Rather, it is dedicated to the analysis of the particular Frame-Cable structure under consideration. This involves the determination of the initial shape of the structure under the application of the gravity load as well as the application of the wind load which is a function of the area and the rotation angle of the elements.

The ATLAS project is sponsored by the Florida Department Of Transportation (FDOT) and it is meant to be used for the analysis of the cable supported signal lights and signs. This dual cable supported system is currently the most commonly used signal light and sign support that is used in the state of Florida. Past experience showed that the current analysis techniques for this type of systems were inadequate. This was evident since this type of systems suffered excessive damages when they were exposed to hurricane or even moderate winds.

7.2 Conclusions

The results of ATLAS were verified by static load tests that were performed at the University of Florida Civil Engineering Structures Laboratory. The results obtained by ATLAS showed a good correlation with the results of the static load tests as shown in Chapter 6 of this dissertation. The tests verified the analytical procedure in the following areas:

1. Initial shape of the structural system which is the shape of the structure under the application of the gravity load.

2. The tensions in the catenary as well as the messenger cable.

3. The catenary as well as the messenger out of plane displacements. These are the displacements that correspond to the wind loads that are applied out of plane.

4. The signal light/sign element rotations. These rotations are used for the calculation of the wind loads on the structure which consist of the drag force and the uplift.

7.3 Recommendations for Future Work

While the ATLAS program provides a powerful tool for the analysis of the signal lights and signs supported by the dual cable system, there is room for future work. Most of this work is related with the modeling of the particular systems. Currently ATLAS approximates the poles as linear frame elements while they are actually tapered. At the same time considers the frame base nodes (foundation) as fixed which is also a very good approximation. A much better model would feature spring supports which are expressed in terms of the soil properties. Another important area of work would be the presentation of data. Currently a graphical preprocessor (ATLASG) developed by the author is used

for the preparation of the input data. The results of the analysis however are presented in data file. A user friendly graphical post-processor would be useful addition to the package.

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BIOGRAPHICAL SKETCH

Petros Marcou Christou was born on July 31, 1967, in Famagusta, Cyprus. He lived in Famagusta for the first years of his life when he was forced to leave the city after Turkey invaded Cyprus in 1974. Now he lives in Larnaca, Cyprus. Upon graduation from high school in 1985, he enlisted in the Cyprus army where he served for 26 months. Then he joined the Frederick Polytechnic in Nicosia, Cyprus, in 1987 where he earned his Higher Diploma in Building Technology. In 1989 he transferred to the University of Mississippi in Oxford, Mississippi, where he graduated two years later with a Bachelors in Civil Engineering. Then he decided to stay in school and join the graduate school. In 1993 he earned his Master of Engineering degree from the University of Florida in Gainesville, Florida, with the specialization in structural engineering. Currently he is still at the University of Florida working towards his Ph.D. degree in civil engineering with the specialization in structural engineering as well.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Marc I. Hoit, Chair
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Ronald A. Cook, Cochair
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Clifford O. Hays Jr.
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Fernando E. Fagundo
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Rick L. Smith
Associate Professor of Mathematics

This dissertation was submitted to the Graduate Faculty of the college of Engineering and to the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

December, 1996

for

Winfred M. Phillips
Dean, College of Engineering

Karen A. Holbrook
Dean, Graduate School